

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

No. 845.

THE LAUNHARDT FORMULA, AND RAILROAD
BRIDGE SPECIFICATIONS.

BY HENRY B. SEAMAN, M. Am. Soc. C. E.

PRESENTED NOVEMBER 16TH, 1898.

In 1889 the author presented, in the form of a discussion on the paper on "American Railroad Bridges,"* by Theodore Cooper, M. Am. Soc. C. E., a review of the fatigue of metals, as demonstrated by the experiments of Wöhler, and of the application of the Launhardt formula to these results. As the paper treated the subject of bridges from a historical, rather than from a technical, standpoint, this portion of the discussion was withdrawn before publication, with the intention of presenting it later in the form of a separate paper. Owing to the pressure of other matters the subject was laid aside until the present time.

In bringing the matter again before the Society, it is proposed to note the most complete of Wöhler's experiments, to repeat the deductions which were made in 1889, and finally to place the matter in such form as to permit of its practical application to bridge construction. For this purpose a general specification for railroad bridges is presented herewith in order to elicit discussion.

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* *Transactions*, Am. Soc. C. E., Vol. xxi, p. 1.

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Pennsylvania Railroad by Joseph M. Wilson, M. Am. Soc. C. E. Later, in the service of bridge shops, he used the old method of dimensioning, as formerly outlined by Mr. Cooper and others; and when subsequently he returned to the use of the Launhardt formula, he recognized more fully the extra labor which its refinements and general application involved, although its results, as being universally applicable to long and short spans and to special conditions, were extremely gratifying where the item of time consumed in calculations was not a matter of importance. It was in the endeavor to simplify the application of this general principle, while yet retaining its technical value, that the original experiments, as published in *Engineering* in 1871, were more carefully examined.

The adoption of the Launhardt formula and the acceptance of the theory of "fatigue of metals" had met with the most strenuous opposition by those who had formulated or used the older methods of design. One of the most severe arraignments of this theory which came to the author's notice, was that of Mr. Cooper,* when he described the term "fatigue of metals" as "absurd and unscientific" and stated that "this result was totally at variance with the acceptance of the perfect elasticity of metals as heretofore accepted," and although his later specifications provide for separate live and dead-load strains in parts of the structure, it is understood that the step was taken because of the general uncertainty in the effect of live load, rather than as any concession to the theory of "fatigue."

To the author's mind, the experiments of Wöhler not only demonstrated clearly the fatigue of metals, but also entirely destroyed the theory of their perfect elasticity, as formerly accepted. These experiments were made with that careful attention to details for which the German Engineers are justly famous, and though they involve such a variety of conditions as to render refined analysis difficult, yet they show a uniformity of results which leads to unquestionable deductions. These experiments have been of the greatest value in the study of the resistance of materials.

WÖHLER'S EXPERIMENTS.

The material with which Wöhler experimented included wrought iron, mild steel, cast steel for axles, and spring steel, but the results obtained upon wrought iron and spring steel were so much more complete and satisfactory than the others that the present review may be confined to them.

* *Transactions, Am. Soc. C. E.*, Vol. xvii, p. 181.

The experiments noted are as follows :

WROUGHT IRON.

Repeated Variations of Tensional Strains (of known amounts).

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
51 360 lbs.	0	800
47 080 "	0	106 910
42 800 "	0	340 853
38 520 "	0	480 852
34 240 "	0	10 141 645
47 080 "	21 400 lbs.	2 373 424
47 080 "	25 680 "	4 000 000

*And not broken.

Repeated Application of Transverse Strain (varying between known amounts).

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
58 850 lbs.	0	169 750
53 500 "	0	420 000
48 150 "	0	481 950
42 800 "	0	1 320 000
38 520 "	0	4 035 400
34 240 "	0	3 420 000
32 100 "	0	48 200 000

*And not broken.

Rotating Bars, Fixed at One End, and Loaded at the Other (giving alternately equal strains in opposite directions).

Test pieces with filleted shoulders.

Strain per square inch.	Number of applications of load before breaking.
34 240 lbs.	56 430
32 100 "	99 000
29 960 "	183 145
27 820 "	479 490
25 680 "	909 810
23 540 "	3 632 588
21 400 "	4 917 992
19 260 "	19 186 791
17 120 "	132 250 000

*And not broken.

Test piece with square shoulder.

Strain per square inch.	Number of applications of load before breaking.
19 260 lbs.	1 603 570
19 260 "	2 063 760
17 120 "	14 695 000

*And not broken.

From these experiments Wöhler recommends the following as the safe limits of strain per square inch:

Maximum strain.	Minimum strain.
— 17 120 lbs.	+ 17 120 lbs.
— 35 310 "	0
— 47 080 "	— 25 680 lbs.

UNTEMPERED CAST STEEL FOR SPRINGS.

(Material furnished by Krupp.)

Repeated Application of Transverse Strains (varying between known limits).

(SET No. 1.)

Maximum strain per square inch.	Minimum strain per square inch.	Number of applications of load before breaking.
107 700 lbs.	0	39 950
96 300 "	0	72 450
85 600 "	0	132 650
85 600 "	0	117 000
74 900 "	0	197 400
64 200 "	0	468 200
53 500 "	0	40 600 000

*And not broken.

(SET No. 2.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
107 000 lbs.	17 762 lbs.	89 238 lbs.	62 000
107 000 "	35 631 "	71 369 "	149 800
107 000 "	53 500 "	53 500 "	400 050
107 000 "	62 381 "	44 619 "	376 700
107 000 "	70 620 "	36 380 "	19 673 300

*And not broken.

(SET No. 3.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
96 300 lbs.	21 400 lbs.	74 900 lbs.	81 200
96 300 "	32 100 "	64 200 "	156 200
96 300 "	42 800 "	53 500 "	225 300
96 300 "	53 500 "	42 800 "	1 238 900
96 300 "	53 500 "	42 800 "	300 900
96 300 "	64 200 "	32 100 "	33 600 000

*And not broken.

(SET No. 4.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
85 600 lbs.	10 700 lbs.	74 900 lbs.	99 700
85 600 "	21 400 "	64 200 "	176 300
85 600 "	32 100 "	53 500 "	619 600
85 600 "	32 100 "	53 500 "	2 135 670
85 600 "	42 800 "	42 800 "	35 800 000
85 600 "	42 800 "	42 800 "	38 000 000
85 600 "	59 920 "	25 680 "	36 000 000

*And not broken.

(SET No. 5.)

Maximum strain per square inch.	Minimum strain per square inch.	Difference.	Number of applications of load before breaking.
74 900 lbs.	10 700 lbs.	64 200 lbs.	286 100
74 900 "	21 400 "	53 500 "	701 800
74 900 "	26 750 "	48 150 "	36 600 000
74 900 "	31 100 "	42 800 "	31 150 000

*And not broken.

From these experiments Wöhler made the following recommendations as the safe limits of strain:

Maximum strain.	Minimum strain.
— 53 500 lbs.	0
— 74 900 "	— 26 750 lbs.
— 85 600 "	— 42 800 "
— 96 300 "	— 64 200 "

These experiments show that the material may be broken by the repeated application of a strain much less than that which would be required to produce rupture by a single application. The five sets of experiments on cast steel each show that as the amount of the strain which is repeatedly applied is decreased, the number of applications required to produce rupture is increased, until a strain is reached which will not produce rupture after an infinite number of applications. Sets Nos. 2 to 5, inclusive, also show in each case that the material may sustain a given maximum strain, but that fracture may be produced by repeatedly removing a portion of this strain. It is evident, therefore, that this change of strain, with the resulting fatigue produced by it, is sufficient to cause rupture under the conditions noted. Could there be any more convincing demonstration that it was fatigue, and fatigue only, which produced the varying results in each set? The opposition to the "fatigue of metals," in the face of these experiments, would seem to be an objection to the use of a word rather than to the acceptance of an idea.

Referring to the tabulated results of the tests on wrought iron, it is not apparent where Wöhler derives the limit which he recommends for all live load in one direction, *i. e.*, — 35 310 lbs. — 0 lbs., since the experiments, both with transverse and with torsional strains, would indicate — 34 240 lbs. — 0 lbs. If the latter limit were substituted for that recommended by Wöhler, the allowable strains would be modified as follows:

— 17 120 lbs. + 17 120 lbs.

— 34 240 " 0 "

— 47 080 " — 25 680 "

Although these results are derived from experiments with different kinds of strain and are too brief to permit of the formulation of any definite theory, there seems ample basis for a general theory of work, *i. e.*, that the material is capable of doing a certain amount of work (derived possibly from the treatment, or work, which it receives in manufacture), and when that amount of work is exceeded, the material becomes fatigued, and its tenacity finally destroyed. Under these circumstances the destructive effect of live strain may be considered as bearing a certain relation to that of dead strain, as suggested by Gerber, and in the experiments on wrought iron just cited, that effect would seem to be about in the ratio of 2 to 1 for all cases, thus:

WÖHLER'S TESTS.		Live strain. (Maximum — Minimum.)	Dead strain.	Equivalent dead strain. (Dead + 2 Live.)
Maximum.	Minimum.			
—17 120 lbs.	+17 120 lbs.	34 240 lbs.	0	68 480 lbs.
—34 240 "	0 "	34 240 "	0	68 480 "
—47 080 "	—25 680 "	21 400 "	25 680	68 480 "

The exact coincidence of these results—as shown in the column of equivalent dead strain—is striking, and although they are too incomplete to be considered as the demonstration of an exact law, yet they may be regarded as confirming the general theory of work.

The experiments on spring steel are equally interesting, and are the most complete and satisfactory of any of those made by Wöhler. They demonstrate conclusively the fatigue of metals; and the fact that they were made upon a homogeneous material makes them of increased value, since any flaw in the material is quickly detected in the results and the misleading test thrown out.

Set No. 1 of these experiments is merely a repetition of the action shown in the experiments on wrought iron. The applied load was gradually diminished for each successive experiment, until one was reached which would not produce rupture after an indefinite number of applications. In the second and following sets a method of procedure was adopted which removed all doubts as to the cause of rupture, as has been already described.

It would hardly seem necessary, in view of these experiments, to go further into the demonstration of the fatigue of metals, but in this connection the results of Bauschinger are so pertinent that they should not be overlooked. By the use of a delicate micrometer he ascertained that, at the very beginning of loading, a slight permanent set was noticeable. How far the element of time affected the result is difficult to determine, since there appeared to be a tendency in the material to return to its former state of rest for days, and sometimes weeks, after the load was removed. The fact of this slight set is probably the true explanation of the results obtained by Wöhler, and it has so modified former ideas of the perfect elasticity of metals, that the term must be abandoned or a new definition sought. Bauschinger himself proposed to use the more correct expression, "Limit of Uniform Elongation," and many engineers have preferred the concise term, "Yield Point," but others still adhere to the old name, "Elastic Limit," trusting to a considerate profession to give it the revised interpretation.

THE LAUNHARDT FORMULA.

These experiments on spring steel are so much more systematic and complete than any others made, that they have been generally regarded as presenting the most accurate information obtainable upon the subject of fatigue of metals, and it was in the endeavor to deduce an expression which would indicate the effect of this fatigue, as compared with that of permanent strain, that Launhardt outlined his formula.

The derivation of the formula* shows that, although it was sought by a rational method of procedure, it contained an arbitrary step which made the value of the formula entirely dependent upon the agreement of its results with those of experiment.

The formula, as outlined by Launhardt, is

$$a = u \left(1 + \frac{t - u}{u} \frac{\text{Min. Strain}}{\text{Max. Strain}} \right)$$

Where a = safe working strength,

u = safe strength for variable load,

t = safe strength for permanent load.

Since the value of this formula depends upon the agreement with experiment; and since the general theory of work suggested will depend upon a similar comparison, it is interesting to note what may be shown in this respect.

In comparing the results of the formula with those of experiment, the value of u was taken from set No. 1 as 53 500 lbs. (500 centners), but the value of t , not having been found in any of the duration experiments made on spring steel, was assumed, and a value of 117 700 lbs. (1 100 centners) was taken, which would bring, by the formula, the intermediate results of sets Nos. 3 and 4 in conformity with those of the tests. Had any other value of t been taken it is evident that the intermediate results would have varied accordingly.

The following table is reduced and corrected from the translation and shows to what extent the results of the formula agree with the experiments made:

Set.	u of Formula. No. 1.	No. 5.	No. 4.	No. 3.	t of Formula.
By experiment (Minimum.).....	0	26 750	42 800	64 200	not found.
By experiment (Maximum.) a	53 500	74 900	85 600	96 300
By formula for a	{ Assumed 53 500	76 480	85 600	96 250	{ Assumed 117 700

* Weyrauch, "Structures of Iron and Steel." Trans. by Du Bois.

It will be noted that in two instances (sets Nos. 4 and 3) the formula gives results corresponding to those of experiment, but since any other value of t than that assumed would have given different results, the comparison can hardly be considered a confirmation of the formula.

Let the general theory of work, as suggested in the experiments on wrought iron, be now applied to these same experiments on spring steel. It has been seen already how strikingly the results agree in iron—too precisely, perhaps, for absolute confidence, since the material is of such variable character—but in the tests with steel the variable results which would naturally occur can be followed more closely. Using the same ratio of 2 to 1 in these results, as was used for wrought iron, the following table is derived :

Set.	WÖHLER'S TESTS.		Live Strain (Maximum — Minimum.)	Dead Strain (Minimum.)	Equivalent Dead Strain (Dead + 2 Live.)
	Maximum.	Minimum.			
No. 1.	53 500 lbs.	0	53 700 lbs.	0	107 000 lbs.
" 5.	74 900 "	26 750 lbs.	48 150 "	26 750	123 050 "
" 4.	85 600 "	42 800 "	42 800 "	42 800	128 400 "
" 3.	96 300 "	64 200 "	32 100 "	64 200	128 400 "

The column of Equivalent Dead Strains in this table shows that the results of sets Nos. 4 and 3 again agree, as they did with the Launhardt formula, but that sets Nos. 1 and 5 fall below them. By examining the tests of set No. 1, it will be noticed that 64 200 lbs. was applied 468 200 times before fracture, and that there was no test between 64 200 lbs. and 53 500 lbs. There is no assurance, therefore, that a weight exceeding 53 500 lbs., and perhaps nearly, or quite, equalling 64 200 lbs., could not have been applied indefinitely. Had 64 200 lbs. sustained the tests satisfactorily, the equivalent dead strain for this set would have been 128 400 lbs., and would have exactly agreed with sets Nos. 4 and 3.

Similarly, in the second test of set No. 5, a maximum load of 74 900 lbs., and minimum of 21 400 lbs., was applied 701 800 times before fracture, and no experiment was made between these results and those of the third test, recommended by Wöhler, of maximum 74 900 lbs. and minimum 26 750 lbs. Had something near the second test of this set been found satisfactory, instead of the third test as recommended, the equivalent dead strain would have been modified, as in set No. 1, to more nearly equal 128 400 lbs., as in the other sets.

Although these deductions are by no means conclusive, they are extremely interesting, and when the results on wrought iron are also considered, this ratio receives much more confirmation than does the Launhardt formula. The "fatigue of metals" is clearly established, a theory of work seems also well founded, and an approximate ratio of 2 to 1 for the relative effect of live and dead strains appears to be sustained by experiment. For the purpose of placing the subject before the Society in a practical form, it has been embodied in the specifications given in the Appendix.

Allowable Strains.—In determining the allowable strains for proportioning parts, a permissible dead strain is selected, and all other conditions are reduced to this basis. Dead and live strains are calculated separately as usual, and proper additions made to the live load for effect of impact. The strains which are resisted by columns may be properly provided for by the column formula, the whole reduced to equivalent dead strain and the required sections calculated in the usual way. This method is applicable to all conditions and locations of members in the structure, and avoids special provisions for local parts as was the common practice in the old method of dimensioning.

An allowable unit strain of 18 000 lbs. per square inch for dead strain may be selected as corresponding to the present practice in steel, though, where desired, this may be increased to 20 000 lbs. without modifying the general method of dimensioning outlined. The experiments of Bauschinger indicate that tensile and compressive strains may be treated alike, except that it is necessary to consider their algebraic signs and the effect of flexure on the outer fiber of long compression members.

In considering the relative effects of live and dead strains, it has been shown that Wöhler's experiments indicate a ratio of 2 to 1, but they were made without intervals of rest and without impact or any other increase of live strain beyond that noted in the tables. In outlining general specifications provision must be made for extreme conditions. In the elevated railways of large cities, or in the large railway terminals, where the passage of trains is almost incessant, there is little rest for the structure; while the vibrations of all rapidly passing loads, together with unbalanced drivers, flat car-wheels and defective track, will increase the live strains very considerably in addition to

the impact due to sudden application. When it is also remembered that according to Bauschinger it takes days, and perhaps weeks, for the material to fully recover from an applied strain, the ratio of 2 to 1 for dead and live strains does not seem excessive.

Having adopted the allowable dead strain per square inch, and accepted the ratio of 2 to 1 for the relative effect of live and dead strains, it is yet necessary to provide for the effect of impact and for the increase of compressive strains due to flexure of columns. The column formula in general use for compression is directly applicable to the proposed method of dimensioning, and may be adopted without change, except that the numerator becomes 18 000 lbs., according to the allowable dead unit strain. For alternate strains, however, the increase of compressive strain on the outer fiber due to flexure of column is found first, and then the compressive strain thus increased is added to the tensile strain, in order to find the total live strain. This increased strain due to flexure may be found by using the reciprocal of the column formula.*

In making provision for impact, there is neither a rational formula nor any complete system of experiments upon which to base a specification, and the practice varies from making no provision at all to making an allowance of 100% for floor system and adjacent connections. Special allowable strains for certain members, such as floor-beam hangers, have been provided in some instances, but it is desirable to avoid all "special cases" in a general specification, and to so frame the general clause as to apply to the elements upon which the special conditions are based. The ratio of 2 to 1 for the allowable dead and live strains may be considered as providing sufficiently for impact on long-span structures not in continuous use. For short spans of 100 ft. or less, additional provision seems necessary on account of the sudden application of the load. To meet this requirement the author would suggest the use of the table contained in the specifications until experiment furnishes authoritative data.

* This is readily explained as follows: The general formula is, $p = P \left(\frac{1}{1 + \frac{l^2}{a r^2}} \right)$ in which P is the greatest strain allowable on any fiber of the column, and p is the reduced direct strain which may be applied at the ends of the column to produce the fiber strain P by flexure. If, therefore, we have given, the direct strain to be applied to the ends of the column and desire to ascertain the resulting increase of strain on the outer fiber, the formula becomes,

$$P = p \left(1 + \frac{l^2}{a r^2} \right)$$

Those familiar with the derivation of the formula will recognize this as retracing one step in its original formation.

Specifications.—The bridge specifications presented as an appendix to this paper, it is believed, correspond to the best practice of the present time. The method of dimensioning conforms more closely to the results of Wöhler's experiments than does the Launhardt formula, and at the same time is direct and simple in its application. The clauses have been made rather full in detail, since it is an easy matter to omit those which are not desired, and it is hoped that the discussion will be so complete that it will enable a fairly uniform general practice to be established, and to that extent to anticipate the work of a committee upon this subject.

The specifications have been written from the standpoint of the railroad company rather than from that of the manufacturer, since it is the company's interests which they are to protect, and since, also, the railroad is later to maintain the structure. It is advisable for every railroad system to have a corps of bridge engineers to design and maintain this class of work, and, when the size of a road does not justify such an organization, several roads may unite in its employment. Railroad engineers would welcome the assistance of a specialist, provided that in so doing they did not jeopardize their own positions.

The specifications provide, either for plans furnished by the railroad company, or for plans furnished by the manufacturer and approved by the company. In either case all original drawings should become the property of the railroad, and for convenience in filing, a standard size of sheet should be adopted.

The extreme refinements necessary for the design of continuous draw-bridges and the impracticability of obtaining the theoretical reactions assumed in calculations has led, in draw-bridge design, to the use of two spans, which may be lifted and revolved, but which, in service, have the direct bearings and reactions of a single span. The use of continuous girders is therefore restricted to those cases where the device of separate spans may be impossible.

It is preferable that the ties, guard-rails, etc., of the floor system be constructed by the railroad company, as men are usually employed for purposes of maintenance and the work will be better and more economically performed by them than by contract, but where this cannot be done the specifications describe the work required of the contractor.

In discussing the paper on "Railway Bridge Designing,"* the author fully reviewed the subject of engine loading. The typical engine here specified corresponds closely to actual conditions. It represents the heaviest engine and train now in general use, though exceptional cases exist which are somewhat heavier, and ore cars are now in service which weigh 4 800 lbs. per lineal foot of track. The loading may be varied to suit different railroads, by increasing, or diminishing, all concentrations simultaneously in the same ratio, as has been proposed by Mr. Theodore Cooper. It is in no case advisable, however, to vary the loading more than 25% either way from that here specified.

The allowable dead strain specified is 18 000 lbs. per square inch, with the usual reduction for compression members. The column formula of Gordon, as modified by Rankine, is used and needs no explanation or endorsement. The impracticability of obtaining columns with fixed ends, in a bridge structure, is the reason for the adoption of only one column formula. In the case of alternate strains the column formula is replaced by its reciprocal,

$$P = p \left(1 + \frac{l^2}{a r^2} \right)$$

as already explained, but alternate strains are to be avoided wherever possible.

Where long unbraced compression flanges are used, the allowable strain is reduced by the Rankine formula,

$$C = \frac{18\,000}{1 + \frac{l^2}{5\,000\,w^2}}$$

but it is so customary to brace top flanges at intervals of about twelve diameters that this formula is rarely used.

The shearing and bearing values of rivets and pins have been reduced to the same basis of allowable dead strain; shear being three-fourths of the allowable unit strain for direct tension, and bearing being one and one-half times the same unit. If the basis of 18 000 lbs. is modified, these values should be correspondingly changed. The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout. The details are the most important part of a structure, and those experienced in

* *Transactions, Am. Soc. C. E.*, Vol. xxvi, p. 227.

maintenance will testify that it is the live strain which causes loose rivets and which wears out the bridge. To provide for dead and live strains in the main members, while making no such provision for the details, is to build a scientific structure upon a crude foundation.

In outlining the details of design the endeavor has been made to cover every material point in first-class work.

The general tendency of recent practice is toward the adoption of soft steel for structural work. Drilling from the solid is obsolete, punching is almost universal, and perfect reaming is only performed when particularly required by the specifications. The injury to steel from rough punching and the danger resulting from flaws increase with the hardness of the steel, and for this reason the soft steel will doubtless continue in preference. The recommendations of the "Committee on Uniform Methods of Testing Materials, etc.," have been generally adopted, except that a milder steel than was there suggested is here specified. Where the rivet holes are punched $\frac{1}{8}$ in. small, and afterward drilled out to full diameter, it might be advisable to use steel of ultimate strength, 60 000 lbs. \pm 4 000 lbs., and elongation 25 per cent. In such case the base for allowable dead strain could be increased to 20 000 lbs. per square inch.

The subject of painting is a matter of controversy. The first essential of a good paint is pure linseed oil; yet it is rarely obtained, and an adulteration is almost impossible to detect. Objection has been made to the practice of giving the steel a coat of oil in the shops, because of the necessity of having the oil thoroughly dry before the paint is applied, but the many advantages of oil for purposes of inspection, together with the fact that the oil usually has several weeks in which to dry before the coat of paint is applied, make it very desirable as a shop coat. For the subsequent coats the author prefers red lead, where the structure is to be exposed to the weather, and a carbon paint where exposed to locomotive fumes. In either case it is important that pure oil should be used. With red lead the difficulties of mixing and applying the paint make it important that only men experienced in this work should be employed.

Upon the matter of inspection the author has little to say. There is no element in bridge construction more important, if properly done, yet none so likely to be neglected. Few appreciate the importance of perfect mechanical work until they maintain the structures they build.

APPENDIX A.

SPECIFICATIONS FOR STEEL RAILROAD BRIDGES.

PROPOSALS AND DRAWINGS.

Each bidder shall submit with his proposal complete strain sheets, showing loads assumed in calculations and the resulting strains, and the sections used. The strains from each kind of load shall be shown separately, and the dead load assumed shall not be less than that of the finished structure. At the same time, each bidder shall submit general plans of the structure, and such details as will show clearly the construction of all members and connections. In case of draw bridges, all machinery shall be similarly shown.

All drawings furnished by the railroad company, whether of general location or of details of construction, shall be strictly followed.

Upon the award of the contract and before work is commenced, a complete set of working drawings, in duplicate, including strain sheets and general drawings previously mentioned, shall be submitted to the railroad company for approval. All material ordered or work done before the drawings are approved shall be at the risk of the contractor.

All drawings shall be made on the dull side of tracing cloth, and of a uniform size of 24 x 36 ins. After the work is completed, these drawings, in good condition, shall become the property of the railroad company for file. The contractor may retain such prints or copies of them as he may desire for record.

GENERAL PROVISIONS.

The structure shall be wholly of rolled steel and wrought iron. A type of truss shall be used in which the strains may be readily calculated and which subjects no member to alternate strains. Continuous girders will be allowed only in case of upper chords carrying floors, and in special cases of draw bridges.

Double-track, through-truss bridges, shall have only two trusses, and four-track bridges only three trusses, unless otherwise specified. In the case of plate girders, special provisions shall be made for spreading the tracks when necessary.

The distance from center to center of double track is 12 ft. 6 ins.

All through-bridges, on tangents, shall have a clear opening as shown on the clearance diagram, Fig. 1.

The width shall be proportionately increased for two or more tracks. On curves, the height and width shall be increased as required by curvature and elevation of rail.

All bridges shall be provided with steel cross-girders and stringers, except in the case of deck bridges, in which the ties may rest on the top chord, and be increased in depth to carry a single track.

Stringers and deck-plate girders shall not be spaced further apart, between centers, than their depth, with a minimum limit of 5 ft.

The wooden floor of ties and guard rails will be furnished and put on by the railroad company, unless otherwise specially provided by contract. The cross-ties shall be of long-leaved yellow pine, 8 ins. square and 9 ft. long, spaced 6 ins. apart in the clear and notched $\frac{1}{2}$ in. over the stringer. In cases where the stringers are spaced more than 7 ft. apart, the depth of the tie shall be increased so that the strain on the outer fiber does not exceed 1 000 lbs. per square inch, considering the weight of a single driver as being carried by two ties.

Guard rails of long-leaved yellow pine, 6 x 8 ins. shall be placed 3 ft. 9 ins. in the clear, from the center of the track. They shall be notched $\frac{1}{2}$ in. over the cross-ties and spliced with a horizontal half and half joint, 6 ins. long, over a tie. They shall be bolted to the cross-tie at the splice, to the cross-tie next adjacent to the splice, and to every fourth tie between, by $\frac{3}{4}$ -in. bolts. To all other ties they shall be fastened by $\frac{3}{4}$ -in. square spikes. Inside rail-guards shall be placed at a distance of 7 ins. in the clear inside the track rail, and shall extend back over the approach to a point not less than 50 ft. from the back wall.

LOADS.

The structure shall be designed to resist the strains from the following loads:

The dead load shall consist of the entire weight of the structure, properly distributed at the various panel points.

The weight of rails, guard rails, splices and bolts shall be estimated at 175 lbs. per lineal foot of track; ties of standard dimensions at 225 lbs. per lineal foot of track, and special ties at $4\frac{1}{2}$ lbs. per foot B. M.

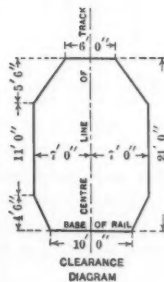


FIG. 1.

The live load shall be the moving load, with impact. The moving load shall consist of two typical consolidated engines, followed by a train, and distributed as shown in the diagram, Fig. 2.

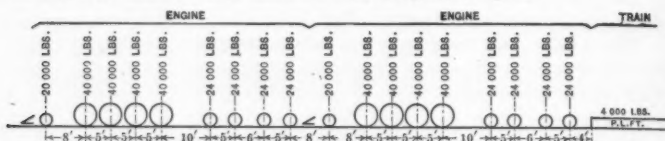


FIG. 2.

There shall be as many trains as tracks, each placed in such position as to produce the maximum strains in the structure.

The impact will be the increase of live strain in any member, due to the sudden application of the moving load, and shall be provided for according to the following table, by interpolation :

L (ft.)	10	20	30	40	50	60	70	80
I (%)	30	20	14	10	7	4	2	1

Where L = Distance in feet through which the moving load must pass to produce the given strain.

I = Percentage of increase of moving load.

When the bridge is on a curve, the lateral bracing and vertical trusses shall be proportioned to resist the centrifugal force due to as many trains as there are tracks, moving in the same direction, at the rate of 60 ft. per second.

Provision shall be made for the sudden starting or stopping of trains, estimating the coefficient of sliding friction at 15 per cent.

Provision shall be made for wind pressure, acting in either direction horizontally : 1st, of 30 lbs. per square foot of the surface of all trusses and the floor, as seen in elevation, considering the ties solid area ; in addition to a train 10 ft. high, beginning 2 ft. 6 ins. above base of rail, and moving across the bridge ; 2d, of 50 lbs. per square foot of exposed surface as specified above, without the train. The wind forces shall be properly distributed between the upper and lower chords.

In the case of plate girders, only one girder need be considered.

Provision shall be made for a variation of temperature of 150° Fahrenheit.

ALLOWABLE STRAINS.

The strains from the dead and the live load shall be calculated separately. The live strain will be the total variation in strain produced by the live load. The dead strain will be the minimum strain to which a member is subjected.

The allowable live strain per square inch shall not exceed one-half of that permissible for dead strain.

The allowable dead strain per square inch for rolled steel shall not exceed 18 000 pounds.

For columns subject to direct compression only, the allowable working strain of 18 000 lbs. per square inch shall be reduced, in proportion to the ratio of length to least radius of gyration, by the following formula :

$$p = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{18\,000 r^2}}$$

Where p = allowable dead strain per square inch, in pounds,

l = length of column, in inches,

r = least radius of gyration of cross section, in inches.

For columns subject to alternate strains of tension and compression, the compressive strain shall be increased, to provide for the increase of strain due to flexure, in proportion to the ratio of the length to the least radius of gyration, by the formula :

$$P = p \left(1 + \frac{l^2}{18\,000 r^2} \right)$$

Where p = direct compression in member,

P = increased strain due to length of column,

l and r = length and least radius of gyration of cross-section, in inches.

The total compressive strain thus found shall be added to the tensile strain, and the resulting strains per square inch shall not exceed those specified for live strains.

In case of compression flanges of beams and girders, the allowable working strains per square inch of such flanges for dead strain shall be computed by the formula :

$$C = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{5\,000 w^2}}$$

Where c = allowable compressive strain per square inch,

l = unsupported length of compressed flange, in inches,

w = width of flange, in inches.

The shearing on pins, rivets, and bolts shall not exceed for dead strain, 13 500 lbs. per square inch of cross-section. Where tension on rivets is unavoidable, it shall not exceed one-half the limit allowed for direct shear. When a force is oblique, the components of direct tension and of direct shear shall be considered separately and the results combined.

The bearing pressure on pins, rivets, and bolts shall not exceed for dead strain 27 000 lbs. per square inch (diameter \times thickness).

In cases of field rivets, driven by hand, an excess of 25% shall be allowed.

Wind pressure, centrifugal force, and sliding friction, either separately or combined, shall not produce greater strain per square inch than that allowed for dead strain.

The pressure, in pounds per lineal inch of roller, shall not exceed $500 \sqrt{d}$ for total load, (d = diameter, in inches).

Bed plates on masonry shall be so proportioned that the greatest pressure on the masonry does not exceed 300 lbs. per square inch.

DETAILS OF DESIGN.

The assumed spans for calculation shall be as follows :

For pin-connected trusses—distance between centers of end pins.

“ riveted girders “ “ “ “ bearing.

“ cross-girders “ “ “ “ trusses.

“ track stringers “ “ “ of cross-girders.

“ cross-ties “ “ “ of track stringers.

The assumed depth for calculation shall be :

For pin-connected trusses—distance between centers of chord pins.

“ riveted girders “ “ “ “ gravity of
flanges (not to exceed the distance out to out of angles).

The whole of the wind force due to the train and floor, and one-half of the truss, shall be considered as acting on the lateral system of the loaded chord, and that due to one-half the truss only, on the lateral system of the unloaded chord.

In the case of deck bridges and very heavy curves, some of the centrifugal force may be transferred to the lower lateral system, in which case the truss shall be duly strengthened. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind strains from the upper lateral system to the end posts, and the end sway-bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal forces from the loaded chord to the abutment.

Each main panel of deck bridges shall be provided with intermediate sway-bracing rods, of a sufficient section to carry one-half the maximum increment due to wind on train, and to centrifugal force. Through bridges shall be provided with post brackets at the intermediate panel points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure, or, when the height of top chord exceeds 25 ft. above base of rail, an overhead system of sway bracing shall be used.

Tension at the windward column of trestle piers shall be avoided if possible, and in any case approved anchor bolts well secured to the masonry shall be used.

The struts shall be proportioned to withstand their component of strain. No reduction shall be made from chord section on account of the material in the lateral system.

All parts shall be so designed that the strains coming upon them may be definitely calculated. The center line of resistance of a member will be along its neutral axis, and connections shall be so designed as to avoid bending, twisting or unequal tearing of the member or its details. The line of strain shall pass centrally through any cluster of rivets which resist it, and where angles or plates are otherwise than so connected, proper provision shall be made for the moments and secondary strains produced. Details will be so designed as to give free access for inspection and painting, and water pockets shall be avoided. In every case the connection of details shall be of greater strength than the member itself.

All members which are subject to direct strains, in addition to bending moments, shall be so proportioned that the algebraic sum of the strains coming upon them shall not exceed the specified allowable strains, properly reduced in case of columns. In continuous upper chords of deck bridges, carrying the floor, the strains due to the live load shall be computed from a bending moment equal to $\frac{3}{4}$ of the maximum moment produced by the engine on a span equal to a panel length considered as a simple beam.

The strain on the outer fiber of solid shapes shall be computed from the moment of inertia of the section.

No allowance shall be made for the web in calculating the flange section of plate girders.

In every case at least one upper flange plate on plate girders shall extend from end to end of the girder, and any additional plates used to make up the flange section shall be made of such lengths as to allow at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of the plates to transmit their value before the theoretical point of the next outside plate is reached. Where the flange plates vary in thickness, they shall decrease outward from the flange angles. Girders formed of web plates and angles alone, having no upper flange plate proper, will not be allowed. The total thickness of plates and angles shall not exceed five times the diameter of the rivet used. Flanges of plate girders over 12 ins. in width shall have at least four rows of rivets, and those over 16 ins. in width at least six rows of rivets.

All flange plates, subject to either tension or compression, spliced in the length of the girder, shall be covered with an extra amount of material equal in section to the material spliced, with sufficient rivets on either side to transmit the strains from the parts cut. Flange angles shall be spliced with angle covers.

In calculating the shearing or bearing strain in web rivets of plate girders, the whole of the shear acting on the side of the panel next to the abutment shall be considered as being transferred into the flange angles at a distance equal to the depth of the girders.

The webs of plate girders shall be spliced, wherever cut, by a plate on each side of the web capable of transmitting the full shearing strain through spliced rivets.

When the thickness of the web plates is less than $\frac{1}{16}$ of the unsupported distance between flange angles, heavy stiffeners shall be riveted on both sides of the web, with a close bearing against the upper and lower flanges and calculated as columns by the compression flange formula for the whole shear at the several points where they are placed. These stiffeners shall be placed at distances, center to center, generally not exceeding the depth of the full web plate, with a minimum limit of 4 ft. Web plates generally shall have stiffeners at bearing points and at points of concentrated loading.

Net sections shall be used in all cases in calculating tension members, and in deducting rivet holes they shall be taken as $\frac{1}{8}$ in. wider than nominal diameter of rivet. In calculating the net sections, having rivets staggered, all rows shall be deducted, unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate.

Rivets shall not be spaced closer than three diameters, center to center, nor further apart, in the direction of the strain, than twelve times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of strain.

Rivets shall not be spaced closer to the side of the plates than $1\frac{1}{2}$ diameters to the center of the rivet, nor further from the side than eight times the thickness of plate. In no case shall the pitch of rivets exceed 6 ins.

Field rivets shall be reduced to a minimum.

Built chords shall be thoroughly spliced with rivets and additional section sufficient to transmit the entire strain; no allowance shall be made for abutting surfaces, except in heavy work.

When necessary to obtain sufficient bearing surface at pin-holes, reinforcing plates shall be added. These plates shall have a sufficient number of rivets to properly distribute the bearing strains from the pins to the member to which they are connected.

All segments of members in compression, connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum strain borne by the segment, and the thickness of

which shall not be less than $\frac{1}{16}$ of the distance between rivets connecting them to the compressed member. In no case, however, shall the length of the batten plate be less than the width of the member.

The distance between connections of strapping shall be such that the individual members composing the column, considered with hinged ends and a length equal to the distance between these connections, shall be stronger than the column as a whole, and in no case shall this distance exceed eight times the least width of these members. Where the ends of the compression member are forked to connect the pins, the strength of each leg shall be at least equal to the entire strength of the column, and the re-enforcing plates shall extend not less than 6 ins. beyond the edge of the batten plates.

Single lattice straps shall have a thickness of not less than $\frac{1}{16}$, and double lattice straps not less than $\frac{1}{8}$, of the distance between the rivets connecting them to the compressed member, and their widths shall be :

For 15-in. channels or 3½-in. and 4-in. angles ($\frac{7}{8}$ -in. rivets).	2½ ins.
“ 12-in. “ “ 3-in. angles ($\frac{3}{4}$ -in. rivets).....	2½ ins.
“ 9-in. “ “ 2½-in. “ ($\frac{1}{2}$ -in. rivets).....	2½ ins.
“ 8-in. “ “ 2-in. “ ($\frac{5}{8}$ -in. and $\frac{3}{4}$ -in. rivets).	2 ins.
“ 7-in. “ “ 2-in. “ ($\frac{5}{8}$ -in. and $\frac{3}{4}$ -in. rivets).	2 ins.

Single lattice bars shall generally be inclined at an angle of 60° to the axis of the member, and double lattice bars at an angle of 45°, with a rivet at their intersection.

All bridges over 65 ft. long shall be provided at one end with turned friction rollers, not less than 2½ ins. in diameter, between two planed surfaces. For spans of 65 ft. or less, planed surfaces shall be used without rollers. The nest of rollers shall be properly protected from the accumulation of dust and cinders.

Trusses shall be secured against side motion on bearing plates and rollers. The bolster blocks shall be joined to the truss, and the bearing plates shall be secured to the underlying supports by bolts or dowels.

Eye-bars shall be so packed as to produce the least bending moment on the pin, and shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in. to 1 ft.

No iron less than $\frac{3}{8}$ in. thick shall be used, except for packing or other idle material. No counter rod shall have less than 1½ sq. ins. of sectional area.

The camber shall be such that under maximum load the bridge will not deflect below a horizontal position.

QUALITY OF MATERIAL.

Rolled Steel.—Rolled steel shall be made by the open-hearth process, and shall contain not more than .04% phosphorus, .04% sulphur, nor .45% manganese. The steel shall be finished straight and smooth, and shall be a perfect product; the slightest flaw will be sufficient cause for rejection at any time during the progress of the work.

The tensile strength, yield point, and ductility of the material shall be determined from a standard test piece of not more than 2 ins. in width, nor less than $\frac{1}{4}$ sq. in. in sectional area, cut from a full-sized bar, and with sides turned or planed parallel, so as to give a uniform minimum section for a length of at least 12 ins.

Whenever practicable, the two sides of the test piece shall be left as they come from the rolls, but the finish on opposite sides shall be alike in this respect.

In determining the ductility, the elongation shall be measured after breaking, on an original length of 8 ins., in which length shall occur the curve of reduction each side of the point of fracture.

The yield point shall be that strain beyond which the elongation ceases to be proportional to the weight imposed, and may be indicated by "drop of beam." It shall in no case be less than 55% of the maximum strain sustained by the test piece. The speed of testing shall be governed by the inspector.

All rolled steel, except rivet steel, shall show by the standard test piece a maximum strain per square inch of 56 000 lbs. \pm 4 000 lbs., with an elongation of 26% in 8 ins. Rivet steel, when tested in specimens of full size of rivet rod, shall show an ultimate strain per square inch not exceeding 56 000 lbs. with an elongation of 30% in 8 ins.

Each melt of finished material shall receive two tension tests—one cut from each extreme variation in thickness of metal rolled. When both tests comply with the specifications, all intermediate thicknesses will be accepted; otherwise only such thicknesses of metal will be accepted as show satisfactory tests.

Each finished piece of steel shall be marked with the melt number.

Wrought Iron.—All wrought iron shall be tough, ductile, fibrous and uniform in quality. It shall be thoroughly welded in rolling, and finished straight and smooth. It shall be free from flaws, blisters, cinder-spots, cracks, and imperfect edges. Scrap steel shall not be used in its manufacture.

The methods specified for testing rolled steel shall apply generally to wrought iron.

All iron shall show by the standard test piece a maximum strength of not less than 50 000 lbs. per square inch and an elongation of 20 per cent.

The yield point, as shown by the standard test piece, shall in no case be less than 26 000 lbs. per square inch.

All iron when cut into testing strips $1\frac{1}{2}$ ins. in width and with corners rounded to $\frac{1}{4}$ in. radius, must be capable of resisting, without signs of fracture, bending cold 90° , with the inner radius not to exceed three times the thickness of the test piece.

All iron which is to be bent in manufacture shall, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without any signs of fracture.

Cast Iron.—All cast iron shall be tough and sound, free from blow-holes, cold-shuts or other injurious imperfections. When broken the fracture shall indicate a good quality of gray iron.

Sample test specimens, 27 ins. long, 2 ins. \times 1 in. in cross-section, cast under the same circumstances as those which attend the casting of the full-sized piece, shall sustain at the center, when resting flatwise upon two dull knife edges, spaced 24 ins. apart, a load of 2 000 lbs.; the load to be sustained two minutes, and show a deflection of not less than $\frac{1}{4}$ in. before fracture.

WORKMANSHIP.

All workmanship shall be first class. All parts exposed to view shall be neatly finished. All nuts shall be hexagonal.

In punching, the diameter of the punch shall not exceed by more than $\frac{1}{16}$ in. the diameter of the rivet to be used, and the diameter of the die shall be as small as may be required to punch a clean hole.

The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than $\frac{1}{16}$ in. will occur. All holes shall be reamed to an exact match before the rivet is driven.

Rivets, when driven, shall completely fill the holes, and shall be machine driven whenever possible. They shall have full, concentric, hemispherical heads of a depth at circumference of shank of not less than one-half the diameter of rivet, and with full bearing on the plates; or they shall be countersunk when so required. Rivet heads shall not be flattened to less than half the diameter of the rivet.

Generally the use of bolts instead of rivets will not be permitted, but when used in special cases the holes shall be reamed parallel and the bolts turned to a driving fit.

Where the work is to be field riveted, the parts shall be assembled before leaving the shops, and the holes reamed to match. In the case of splices of upper chords, or other compression members, they shall be brought to forcible contact by the use of turnbuckles, and after reaming shall have match marks put on the pieces so that they may be brought to proper position in the field, before riveting.

Finished members shall be true and free from kinks, twists and open joints.

Rods and bars which are to receive a thread shall be properly upset before the thread is cut. Where threads are cut on steel, they shall be properly filleted.

All members requiring adjustment shall be provided with sleeve nuts and check nuts. Open turnbuckles will not be allowed. The ends connected shall be distinctly punch-marked, at a distance of 12 ins. from the screw ends, so that these ends may be accurately located inside the nut.

Heads of eye-bars shall be of sufficient section to break the bar in every case. Bars shall be full size at the neck; no patching will be allowed.

Welds in the body of the bars or rods will not be allowed.

Pins shall be turned, perfectly finished, and straight.

All members having bearing on pins shall be carefully bored at right angles to the axis, unless otherwise shown in the drawings. No variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{16}$ in. For pin holes, in pieces which are not adjustable for lengths, no variation of more than $\frac{1}{4}$ in. in length between centers of pin holes will be allowed.

Eye-bars shall be perfectly straight before boring; the holes shall be in the center of the heads and on the center line of the bar. Bars which belong to the same member shall be bored at the same temperature and in one operation. They shall be marked for erection, so that they may be used in the same member.

When bolts are used instead of pins, a variation of $\frac{1}{16}$ in. will be allowed between diameter of bolt and hole.

All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned perpendicular to the direction of the strain, so as to insure even bearings.

Stiffeners of plate girders shall be milled to fit tightly against the flange angles, and shall be packed straight. The packing shall fit close to the flange angles, leaving no open space.

Rollers shall be turned and roller beds and bed plates planed; the bottom of shoes shall be planed exactly parallel to the center line of pin, unless otherwise shown in the drawings.

Thickening washers shall be used whenever required to pack the pin joints tight.

Pilot nuts shall be furnished for each size of pin to preserve the thread of the pin, and to facilitate erection.

PAINTING.

All iron shall be scraped free from scale, and receive one coat of pure kettle-boiled linseed oil before leaving the shops, and one coat of approved paint after erection. All surfaces which come in contact,

or are enclosed, shall receive one coat of approved paint before being assembled.

All turned or faced surfaces shall receive a coat of white lead and tallow before leaving the shops.

INSPECTION.

Free access and information shall be given by the contractor for a thorough inspection of material and workmanship.

The inspector will make detailed reports of his inspection to the engineer, and may notify the contractor of any defects in material and workmanship, but all acceptances made by him shall be considered as temporary, and his inspection shall in no way relieve the contractor of full responsibility for the character and accuracy of the work until its completion and final acceptance by the engineer.

The contractor shall furnish without extra charge such standard test pieces as may be necessary to determine the uniform quality of the material, and also the use of a reliable testing machine with necessary labor for testing.

Full-sized members may be tested to destruction by the engineer. They will be paid for at cost, less their scrap value, if they fulfill the requirements, but when the requirements are not complied with, the tests shall be at the expense of the contractor. Such rejections of the finished material as he may consider warranted by the results of these tests will be made by the engineer.

DISCUSSION.

Mr. Breithaupt. WILLIAM H. BREITHAUP, M. Am. Soc. C. E.—During the years 1859 to 1870 Herr A. Wöhler, a mechanical engineer on railroads, made for the Prussian Government a series of minute and careful experiments on the effect of often-repeated straining of iron and steel, by which he established what is known as Wöhler's law, which is an expression of what is characteristically called the fatigue of metals. Wöhler's conclusions were confirmed by Professor Spangenberg, who continued the investigation after 1870. The facts ascertained were promptly made use of by Gerber, Launhardt, who established formulas since known by his name, and others.

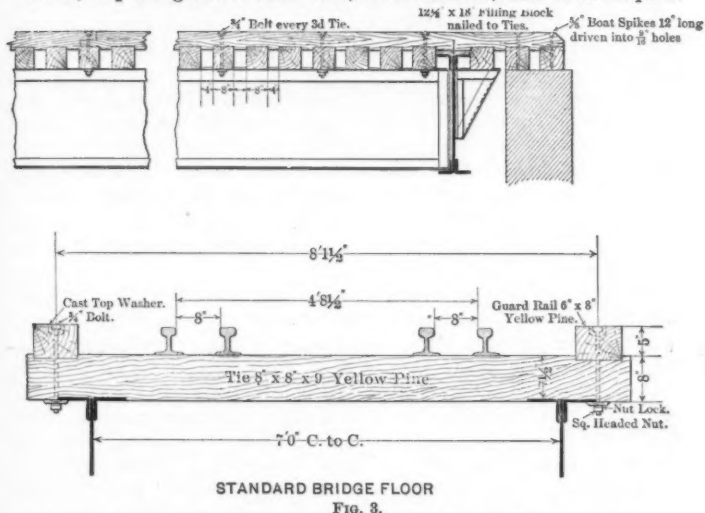
The author, in his clear analysis of Wöhler's experiments, deduces a basis of proportioning, and leaves little ground for objection to the theory of the fatigue of metals. As to the basis of proportioning, there is still difference among engineers. In the opinion of the writer, fixing the live-load allowable unit stress at only half that of the dead load gives a result which is too low. Bauschinger demonstrated, as referred to by the author, that, within limits, material does recover from strain, and it must be assumed that this recovery is more or less according to the time allowed for it. Wöhler made applications of stress at the rate of 72 per minute. Even at this rate constantly maintained, it will be noted that it takes over a year to make 40 000 000 repetitions. Such conditions obtain in machines, but not in structures. It should be said here that of Wöhler's experiments a few isolated ones were on the effect of variation of time intervals between applications of stress. In two sets of bending tests, the specimens being held at one end only, the results were as follows:

Maximum stress in pounds.	NUMBER OF APPLICATIONS TO BREAK SPECIMEN.	
	18 per minute.	72 per minute.
29 960	170 900	183 145
25 680	610 000	909 810

Even with the longer intervals the applications are so frequent as to exclude any idea of recovery of material. The differences in the number of repetitions do not agree proportionally, and at best these few experiments are not conclusive. Wöhler himself modified his coefficients for cases of maximum loading applied only at long intervals. Launhardt's formula gives smaller sections than does the author's method, and so do the unit stresses adopted by other engineers, although in general not differing much.

The author's elimination of continuous drawbridges can apply Mr. Breithaupt economically, as no doubt it is meant to, only to turn-table drawbridges of shorter span. Instead of the latter, single or double-leaf bascule bridges, as now developed, are in most cases preferable.

The author fixes the distance from center to center of track on double-track bridges at 12 ft. 6 ins. It should be whatever it is on the road. Ten years ago 12 ft. between centers on double track was common, now it has been increased to 13 ft. on a number of roads. There should be no change in alignment on or near a bridge if practicable to avoid it. Neither should there be change of grade. An abrupt change of grade is a fruitful cause of the parting of freight trains, of pulling-out of draw bars, of derailment, and of consequent



injury or wrecking of structures. There is naturally a down grade toward watercourses, and the resulting changes of grade should be eased, and located as far away from the bridge as practicable.

If a single driver axle of the specified weight is carried by two 6 x 8-in. ties only, the extreme fiber stress per square inch with 7-ft. stringer spacing will exceed 1 000 lbs. by more than 400 lbs. With an 80-lb. or even a 75-lb. rail, it is usual, and is allowable, to consider the axle as carried by three ties. For a stiff floor, reasonably continuous in case of derailment, the ties should be spaced not more than 4 ins. apart in the clear. The guard-rails should have an even surface, and not be obstructed by washers or bolt heads. The desirability of this is obvious in considering derailment, or in the case of anything

Mr. Breithaupt. dragging on the guard-rail which would be caught and thrown about by projections thereon. Over the floor beams (a space of 12 ins. or more), some support to the rail is desirable to avoid undue stress in the adjacent ties. This can readily be given by a filling block nailed in. A bridge floor as specified by the writer is shown in Fig. 3.

The live load given by the author is a typical one as used in present designing. The percentages for impact give increase for floor proportioning. Another method used* is to assume a uniformly heavy load over the entire bridge, with one local concentration only. Very heavy cars are to some extent coming into use, as referred to by the author, but in any event it must be assumed that engine loads will continue to be heavier than train loads, so that for a length of 100 ft. at the head of the train the load will be heavier than for the rest of the train. Proportioning on the basis specified by the author is therefore more rational.

On the subject of impact there is considerable agitation at present, and it is to be hoped that we may have more definite results. The author well says that there is as yet little or nothing on which to base definite specifications. It is, however, difficult to see why, if for 40-ft. spans impact is taken at 10%, it should be only 1% for 80-ft. spans. The writer has specified for impact as follows: 20% for stringers, 15% for floor beams, and for main girders in the proportion of 10% for 40-ft. spans to 5% for 100-ft. spans.

In riveting there are a number of things that affect the value of the rivet, such as slight mismatching or misfit of holes, short or over-length of rivet, under-heating, eccentric or otherwise careless driving, etc.; on the other hand, their value is largely added to by friction between the surfaces they hold together. In many cases the number of rivets is determined by their permissible minimum spacing. For field rivets, which are usually hand-driven, and the bulk of which are for floor connections, with a large excess of live-load stress, a proportional allowance on the side of safety is made. Nevertheless, the author's method of proportioning for rivets and details throughout in the same general way as for main members is the more rational one.

Unit dead-load stresses are given for shear and bearing on pins, rivets and bolts which are correspondingly somewhat higher than those allowed by most engineers. But there is no special value given for bending stress in pins. If the general dead-load unit stress is taken for this, as seems the intention, it will be inconsistent with the values given for shear and bearing, and furthermore, will give unnecessarily large pins.

The author is to be commended for not specifying any difference in unit stress between plates and shapes and eye-bars. It has

* Instance, Pennsylvania Lines West of Pittsburg Specifications.

been repeatedly shown* that shapes, when properly connected, are as good for net section as are eye-bars, if not a trifle better on account of the annealing of the latter.

The pressure on rollers in pounds per lineal inch is specified as not to exceed $500\sqrt{d}$. With the minimum dimension of rollers specified, this does not apply for short single-track spans; for longer spans it is a small value. It has been found by experiment† that $1200\sqrt{d}$ lbs. is entirely safe, and this has appeared in bridge specifications. A common specification for this value is $300d$.

Provision for expansion ends does not generally receive proper attention. Frequent observation of dislocation of expansion rollers, in long and short spans indifferently, by the accumulation of dust or sand blown in between the rollers, has led the writer to adopt sliding expansion ends for spans up to 90 ft., and, for roller ends, ribbed bed-plates, made of small I-beams or of matched rails of a well-known pattern, having, under the roller surface, longitudinal recesses, open above; and to give all expansion bed-plates, whether for sliding or rolling, central guide ridges fitting into grooves on the rollers.

In conclusion, the writer does not understand why there should not be agreement and greater conciseness among engineers in the use of two certain words. To convey the same meaning, Stoney, an old and still valuable authority on many things, used the word "strain"; Rankine, later, used "stress"; Du Bois, of present American writers, uses or has used both "strain" and "stress"; Burr uses "stress," Johnson "stress," and so on. Most engineering writers, of text books, at least, if not of specifications, now agree in the use of "stress." Among engineers, generally, there is not such uniformity, and in engineering periodicals the confusion is apparent. Within the past two weeks one of the latter, in general notable for good matter and language, said "strain," and elsewhere in the same number "stress," for expression of identical meaning. Rankine, than whom no one is more entitled to fix engineering nomenclature, adopted the word "stress" for force applied to a body, and "strain" for alterations, of what nature soever, in the volume and figure of a solid body, produced by forces applied to it.‡ Under this definition, then, there is no such thing as strain per square inch as little as there is stress in length or width, or in the internal alterations of arrangement of the molecules of the body. There are many uses of "strain" to which, even in popular language, "stress" does not apply at all. Strain and straining are very convenient general terms for torsion, bending, tension or compression. The autographic record of such

* "Recent Tests of Bridge Members," by J. E. Greiner, *Transactions, Am. Soc. C. E.*, Vol. xxxviii, 1897, p. 41.

† Crandall, Cornell.

‡ Rankine, "Applied Mechanics."

Mr. Breithaupt. an apparatus as the old Thurston testing machine, or of the extensometer—for which strainometer would be a better name, since it measures both extension and compression—is a strain diagram or strain sheet, while a diagram of a structure showing the forces applied to the various members is a stress sheet.

Mr. Merriman. MANSFIELD MERRIMAN, M. Am. Soc. C. E.—The speaker agrees with one or two of the written discussions in which it is claimed that the elastic limit is a more suitable point from which to reckon working stresses than the ultimate strength. The lowest ultimate strength allowed for steel by the author's specifications is 52 000 lbs. per square inch, and the lowest elastic limit about 28 600 lbs. per square inch. The latter figure indicates the degree of security afforded by the specified working stresses in a very much better manner than the former.

Wöhler's experiments determined smaller ultimate strengths under a large number of repeated loads, and Launhardt's formula is merely a convenient method of expressing these values after division by a factor of safety. Without a knowledge of the elastic limit, however, it is difficult to arrive at a sure decision whether Launhardt's values are or are not too high. Whether the stresses be steady or repeated, the elastic limit is to be regarded as a constant quantity, provided that none of the stresses surpasses that limit.

It is very much to be desired that an extended series of tests, like those of Wöhler, should be undertaken in accordance with modern laboratory methods. The author, in his discussion of Wöhler's results, appears to have found difficulty in ascertaining the tensile strength of the metal under a steady pull. This would not be the case in any future series of experiments, as specimens from the same bar would be tested under both steady and repeated loads. The tests carried on at the Watertown arsenal since 1888 by Mr. James Howard fulfill all requirements as to care and precision, but they cover only the case of rotating bars under transverse stress. They furnish no data for comparison with Launhardt's formula, since the fiber stresses alternate equal distances in tension and compression, but with respect to the question of fatigue they appear to harmonize with Wöhler's conclusions.

In one of these Watertown tests cast iron showed a remarkable power of resistance to fatigue. The specimens were gun iron, having an ultimate tensile strength of 31 200 lbs. and an apparent elastic limit of about 13 000 lbs. per square inch. Under alternating fiber stresses of 25 000 lbs. per square inch, applied at the rate of 400 repetitions per minute, rupture occurred after 4 700 repetitions. Under 20 000 lbs. per square inch rupture occurred after 26 400 repetitions. Under 15 000 lbs. per square inch rupture occurred after 47 283 500 repetitions. It is difficult to draw general conclusions from a single series of experiments like this, but it certainly shows that this cast

iron, under alternating stresses very near to its apparent elastic limit, Mr. Merriman. had a resistance to fatigue almost, if not fully, equal to many of the steel specimens under similar conditions.

The author's discussion of the question of fatigue seems unnecessary since the stresses in the members of a bridge truss should be kept below the elastic limit, and hence fatigue cannot result. In fact, the ultimate strength as determined by a steady pull appears to have a far greater value than the ultimate strength as found from an enormous number of repeated stresses, provided the actual working stresses are kept below the elastic limit. In respect to high and unusual stresses caused by unbalanced locomotive drivers moving at high speed, or by derailed trains, there may be a slight amount of occasional fatigue; but these stresses should properly not be taken into account in general rules applying to the normal and usual stresses.

The speaker does not wish to be understood as objecting to the author's rule which makes the effect of a computed live-load stress double that of a dead-load stress. If the live load be applied with perfect suddenness, this rule has a sound theoretical basis, and, although the factor 2 is theoretically too large for railroad trains, yet it is a good figure to use in order to provide for contingencies and for future increase in live loads. The author's general method of reducing all stresses to an equivalent dead-load stress, and of using the single value, 18 000 lbs. per square inch, as the allowable unit stress, seems a good and logical one.

The percentages allowed for impact, in the case of members whose maximum stress is attained by a movement of less than 80 ft. in the live load, are unfortunately arbitrary rather than experimental. Moreover, in counters it makes some difference whether these percentages are added to the computed stresses or to the live load, the former being implied in the first sentence of the third paragraph on page 156, and the latter in the statement that I is to be applied to the moving load.

The author uses the column formula in a very rational manner, the numerator P in the expression

$$p = \frac{P}{1 + q \frac{l^2}{r^2}}$$

being regarded as the true measure of the internal stress on the concave side of the deflected column. The speaker has urged this view for many years, having presented it in the first edition of "Mechanics of Materials" in 1885. The value of q properly depends upon the end conditions of the column; but, as the author's specifications make no distinctions in this respect, $\frac{1}{18000}$ seems a fair general value, particularly when it is considered that $\frac{l}{r}$ is never very large in chords and

Mr. Merriman. end posts. In a steel column with fixed ends the speaker regards $\frac{1}{24}$ as a good value for q , while in a column with one end fixed and the other hinged $\frac{1}{14}$ seems preferable.

It is unfortunate, perhaps, that in discussing a paper of this kind the temptation to criticism is stronger than the inclination to approve and commend. Hence, the speaker desires to say that he considers the specifications of the author clear, concise, carefully considered and well adapted to produce bridge structures of good workmanship and of ample security.

Mr. Schneider. C. C. SCHNEIDER, M. AM. SOC. C. E.—While reading that portion of this interesting paper, relating to the Launhardt formula, the speaker was reminded of a sentence in a letter by the late Professor Winkler, of Berlin. That letter is dated Jan. 19th, 1888, more than ten years ago, and in it he says:

"We have at present our trouble to abolish the errors which were caused by the abuse of Wöhler's experiments in determining permissible working strains, since Bauschinger has proved that repeated strains have a detrimental influence only above the elastic limit and alternate strains up to near the elastic limit."

It seems, however, that, in spite of Professor Winkler's efforts, these errors have not yet been eradicated. The experiments selected by the author do not appear to prove either the fatigue of metal inside the elastic limit, or his theory that a live-load strain produces twice the effect of a dead-load strain.

Wöhler's duration experiments were made for the purpose of determining the ultimate strength of the material, under repeated strains in one as well as in alternate directions. Based on these experiments, many elaborate theories were advanced by theorists and conveyed into more or less complex mathematical formulas, to be used for determining permissible working strains, the theorists entirely losing sight of the fact that their theories were based on results above the elastic limit to be used for conditions within the elastic limit, as they exist in actual practice.

The well-established theory of the elastic line is based on strains below the elastic limit, *i. e.*, the limit within which the elongation is proportional to the strain. As a single strain above the elastic limit produces a permanent set and destroys the property of uniform elongation in the metal, the effect of a single permanent strain is not different from the effects of repeated strains, as the single strain has practically destroyed the usefulness of the material and will ultimately produce rupture, if repeated often enough. The elastic limit, therefore, is actually the ultimate strength, for all practical purposes. Tests of material for rupture should be used only for the purpose of establishing its quality, and not for deducing laws or formulas for working strains for either tension or compression members.

The author has selected three of Wöhler's experiments on wrought

iron by which to prove his theories. The first one appears to prove Mr. Schneider. that repeated alternate strains of equal intensity in both directions produce twice the effect of repeated strains in one direction.

The second experiment proves that repeated strains of one kind, within the elastic limit, have no more detrimental effect upon the metal than a single strain.

The third experiment, on which the author relies particularly to prove that a live-load produces twice the effect of a dead-load strain, appears to be of no value for the purpose of deducing a law, as the maximum strain to which the metal was subjected was 47 080 lbs. per square inch (very nearly the ultimate strength). This would practically destroy the material on the first application, and the experiment was continued on material which had already lost its property of uniform elongation.

The author does not consistently follow his own theories, as he considers wind strains to be dead strains, which, according to his own definition, they are not.

Bauschinger's very carefully conducted and reliable experiments, with metal subjected to repeated and alternate strains, appear to prove that the difference between the elastic limit for alternate strains of equal intensity in both directions and repeated strains in one direction is far less than was generally accepted on the strength of the few experiments made by Wöhler.

In Table No. 1 the results of some of Bauschinger's experiments on the resistance of various kinds of metals to repeated and alternate strains are given. Therein may be found the ultimate strength or strains which produced rupture in one application, the strains which resisted an infinite number of repeated applications and the strains which resisted an infinite number of repeated alternate applications of the same intensity in both directions. The values given are in pounds per square inch, and are approximate only.*

TABLE No. 1.

	Ultimate strength.	Repeated strains.	Alternate strains.
1. Wrought iron.....	49 500	28 450	25 170
2. Ingot iron.....	62 000	34 140	28 160
3. Material not specified.....	57 600	31 200	28 160
4. " " ".....	57 180	34 130	32 140
5. Thomas steel.....	87 050	42 670	42 670
6. Rail steel.....	84 480	39 820	39 820
7. Boiler plate, ingot iron.....	57 600	34 130	27 000
8. Material not specified.....	47 650	31 200	22 750

* Föppel's Technische Mechanik.

Mr. Schneider. These experiments indicate that alternate strains reduce the elastic limit below that established by strains in one direction, and, if the working strains were kept within that lower elastic limit, it would not be necessary to consider the strains in the opposite direction, and, therefore, the absolute maximum strain could be used in determining the section of a member.

All the results of experiments appear to prove conclusively that within the elastic limit all strains have the same effect; the internal strain of a member being proportional to its elongation or reduction. It is therefore evident, as far as the resistance of the material is concerned within the elastic limit, that it makes no difference whether this strain is produced by the weight of the structure, the static effect of a superimposed load, or the dynamic effect of a moving load.

A structure is safe if it is able to resist the exterior forces acting upon it for an infinite period. To accomplish this all members of the structure should be so proportioned that the material composing them will be able to resist permanently the strains produced by the exterior forces.

Now the question arises: What are the exterior forces acting upon a railroad bridge? They are:

1. The weight of the structure.
2. The static effect of the moving load.
3. The dynamic effect of the moving load, commonly called impact.
4. The strains produced by wind, centrifugal force and momentum of train.

As experiments have shown that strains in the members of a bridge produced by a fast-moving train are greater than those produced by the static load of the same train, it is necessary to recognize the fact that the strains are increased by the dynamic effect or impact of the moving engine and train. This fact has also been recognized by engineers and considered in specifications.

Some of the older specifications required that a percentage for impact be added to the live-load strains for stringers, floor beams and hangers. The speaker believes that Joseph M. Wilson, M. Am. Soc. C. E., was the first one in the United States who considered the impact for all spans, in his specifications, published in 1885.* These made it dependent upon the proportion of live to dead load, for which purpose a formula is given, which, though similar to the Launhardt formula in form, is, however, different in the results.

Many other engineers have followed Mr. Wilson's example.

Theodore Cooper, M. Am. Soc. C. E., in his specifications for railroad bridges allows for live load only half the permissible working strain he does for dead load, which is practically the same as using a uniform working strain for all loads and adding 100% for impact to all live-load strains, irrespective of the length of the span.

* Transactions, Am. Soc. C. E., Vol. xv, p. 390.

The speaker in his practice uses a uniform working strain, and increases the live-load strains by adding a percentage for impact to the strains produced by the static effect of the live load, making the effect of impact dependent on the length of the span.

The author also considers the dynamic effect of the moving load, and makes provision for impact in spans up to 80 ft.

If impact is considered an established fact, as the author evidently does, then it must be admitted that it exists in all spans.

As the experiments made, up to the present time, do not give sufficient data for computing the dynamic effects of the live load with accuracy, it is necessary to use some practical judgment in this respect and to allow ample margin so as to be certainly on the side of safety.

If reference is made to the speaker's specifications, and the percentages of live load assumed for impact are compared with the experiments made by Professor Fränkel, S. W. Robinson, M. Am. Soc. C. E.; J. E. Greiner, M. Am. Soc. C. E.; F. E. Turneaure, Assoc. Am. Soc. C. E., and others, it will be seen that such percentages are far in excess of the values found by the experiments. The speaker has already given his views on this subject in the discussion of Mr. Greiner's paper entitled, "What Is the Life of an Iron Railroad Bridge?"* and his reasons for adopting the impact method.

As most specifications in use at the present time make provisions for increased strains due to the dynamic effect of the live load in some way or other, either by disguising it in the shape of a formula, or by allowing different working strains for dead and live load, the speaker believes it would be more rational and logical to consider the dynamic effect of the live load separately, and, if in the future more reliable data for computing the impact should be obtained the numerical values of the coefficients could be changed, which would, however, in no way affect the principle of the system.

The speaker fully agrees with the author, that a uniform working strain should be used for all stresses, that strains allowed for tension should be the same as those for compression, and shearing and bearing strains in proportion; or, in other words, one square inch of section should represent a certain number of pounds of strain. This seems to be the most rational method which will insure the proper proportions of details and connections in relation to the main members of a structure. The speaker has used this method in his practice for the last twelve years and has found it to work very satisfactorily.

The author specifies a working strain of 18 000 lbs. per square inch; the speaker uses in his practice a working strain of half the

* *Transactions, Am. Soc. C. E.*, Vol. xxxiv, p. 828.

Mr. Schneider. elastic limit, or a factor of safety of two. This would give, for the steel specified by the author, with an average elastic limit of about 30 000 lbs. per square inch, a working strain of 15 000 lbs. per square inch.

The method proposed in the author's specifications for proportioning the members of railroad bridges, may appear entirely different from the speaker's practice, the author multiplying the live load by two, adding an additional variable percentage of impact for spans up to 80 ft., and using a working strain of 18 000 lbs. per square inch, while the speaker adds a variable percentage for impact to the live load for all spans, using a working strain of 15 000 lbs. per square inch. However, if the results are compared, only small differences will be found in the main members of the bridges. The greatest differences occur in the lateral systems. The assumed wind strains being the same in both specifications, the difference consists in the specified working strains. The author allows 18 000 lbs. per square inch, while the speaker allows only 15 000 lbs. per square inch, making his lateral bracing 20% stronger than if computed in accordance with the author's specifications.

Table No. 2, computed by the speaker, gives the tensile strains per square inch, in the chords for different lengths of spans, produced by the total load, calculated in accordance with both specifications.

TABLE No. 2.

Length of span in feet.	STRAIN PER SQUARE INCH IN POUNDS.	
	Seaman.	Schneider.
10.....	7 170	7 850
50.....	9 120	8 660
100.....	10 380	9 680
200.....	10 870	10 760
300.....	11 270	11 540
400.....	11 640	12 150
500.....	12 100	12 680

On page 152 the author makes the following statement, to which the speaker takes exception:

"The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout."

There were specifications in existence for many years which covered that point. In 1887 specifications were published by the Pencoyd Iron Works which made this distinction between live and dead-load strains in proportioning details without the use of the Launhardt formula.

Referring to the specifications proper, the speaker believes that if Mr. Schnelder, bridge work were done in accordance with these specifications first-class material would be obtained, but only average designs and workmanship. While the speaker agrees with the author in a general way, and is pleased to notice that the specifications are short and precise without any useless requirements which would only increase the cost of the work without improving its quality, they do not, however, come up to the speaker's ideas of the best modern practice as regards design and workmanship.

These specifications will not compel the manufacturer to make first-class designs, unless he is led to do so on his own account, because he takes pride in his work and has ability, experience and sufficient judgment, which the speaker regrets to be compelled to admit is not always the case. The same remark refers also to workmanship.

Under "General Provisions," the author says: "A type of truss shall be used in which the strains may be readily calculated, and which subjects no member to alternate strains."

This clause would prohibit a class of structures which in late years has given much satisfaction to engineers who have had charge of the maintenance of bridges, viz., the riveted-truss and lattice bridges. There are some riveted-lattice bridges in existence which were built over twenty-five years ago in which the workmanship is very poor, and yet they are doing good service at the present day; while pin-connected spans, built at the same time under similar conditions and specifications, have been replaced by new structures years ago. If experience counts for anything, the riveted-lattice bridge has certainly held its own; in fact, modern practice requires riveted bridges for spans greater than those for which plate girders are considered practicable, and up to those where pin-connected spans are advisable, generally between 100 and 130 ft. The speaker, therefore, is of the opinion that the portion of this clause referring to alternate strains should be omitted.

The author specifies: "Stringers and deck-plate girders shall not be spaced further apart, between centers, than their depth, with a minimum limit of 5 ft." There appears to be no good reason for specifying that limit, excepting that it was formerly the practice on the Pennsylvania Railroad. It was abandoned years ago. For double-track bridges the usual practice is to space the stringers $6\frac{1}{2}$ ft. apart between centers (the distance between centers of tracks being 13 ft.), which, in the speaker's opinion, should be the minimum distance between centers of stringers or deck-plate girders.

Under "Loads" the author specifies that provision shall be made for the sudden starting or stopping of a train, estimating the coefficient of sliding friction at 15 per cent. The almost universal

Mr. Schneider. practice is to use a coefficient of friction of 20 per cent. As in some cases the train may be brought to a sudden stop by reversing the engine and applying sand to the rails in addition to using the brakes, in which case the coefficient of friction would be over 30%, the writer would recommend adherence to the usual practice.

In the formula for compression members the author uses a coefficient of 18 000 in the denominator. The speaker would suggest 10 000 instead, so as not to encourage long, slender compression members, and for the same reason would recommend the following additional clause:

"No compression member shall have a length exceeding 100 times its least radius of gyration, excepting those for wind bracing, which may have a length not exceeding 120 times the least radius of gyration."

The speaker would also recommend that the least width of posts in pin-connected bridges be limited to 12 ins., in order to offer some resistance in case of a derailment. The formula for members subject to alternate strains is somewhat confusing, as in this formula p denotes the direct compression in the member, while in the formula for members subject to compression only, p denotes the allowable strain per square inch. Instead of using a complex formula, practically the same results would be obtained by substituting the following clause:

"Members subject to alternate strains of tension and compression shall be so proportioned that the total sectional area is equal to the sum of the areas required for each strain."

The author limits the working strain on rivets in tension to one-half the limit allowed for direct shear. As the author specifies steel for rivets, and as recent experiments have shown that steel rivets in tension give just the same results as round bars of the same diameter, invariably breaking in the shank, there appears to be no good reason for such a low limiting strain. The speaker would suggest that the allowable tensile strain on rivets be made the same as that for direct shear.

The author has omitted to specify the working strain allowed for bending on pins, which, in accordance with good practice, may be the same as that for bearing.

The author specifies that expansion rollers and bed plates should be proportioned for the total load. If the effect of the live load is different from that of the dead load, why should this not apply also to rollers and bed plates?

Under "Details of Design" the author specifies that: "Each main panel of deck-bridges shall be provided with intermediate sway-bracing rods." Modern practice requires stiff bracing.

Referring to trestle towers, the author says: "The struts shall be proportioned to withstand their component of strain." This evidently refers to trestle towers with adjustable rods. The speaker is not aware of any first-class railroad which would accept trestle towers with

adjustable rods at the present time. This should, therefore, be considered antiquated construction and not in accordance with the best modern practice. Mr. Schneider.

The author specifies: "Girders formed of web plates and angles alone, having no upper flange plate proper, will not be allowed."

There appears to be no good reason why girders without an upper flange plate should not be allowed, as all theoretical and practical considerations point the other way, excepting that it was once the practice on the Pennsylvania Railroad. This restriction has been abandoned by this railroad as well as by most other railroads which followed that practice, as being a nuisance to the Maintenance of Way Department. The general tendency of modern practice seems to be to allow no upper flange plates on girders which carry ties on the upper flange, as far as this is practicable.

The speaker thinks the following clause should be omitted: "Flanges of plate girders over 12 ins. in width shall have at least four rows of rivets, and those over 16 ins. in width at least six rows of rivets," as in his experience he has had occasion to observe that this very clause, being put in the specifications as a cast-iron rule without qualification, has produced in some instances very ridiculous results.

In accordance with the author's rule for proportioning plate girders, a girder 24 ins. deep with a web plate $\frac{5}{8}$ in. thick would require stiffeners every 2 ft. In the speaker's opinion the intermediate stiffeners in a girder of such proportions might be dispensed with.

The author's rule for determining the length of tie-plates at the ends of compression members, whose segments are connected by laticing, seems to be made to accommodate the poorest workmanship, as even moderately good workmanship (as long as the pin is bored at right angles to the center line of the member) will distribute the pressure over both segments. Strict conformity to this rule, which is also found in some other specifications, has produced in some cases very anomalous results. The speaker is of the opinion that if the end tie-plates are made as long as the width of the member, it will be sufficient for all cases.

The author specifies that: "All bridges over 65 ft. long shall be provided at one end with turned friction rollers, not less than $2\frac{1}{2}$ ins. in diameter."

The speaker would suggest that for plate girders of ordinary length, that is, up to 80 ft., the expansion end should slide on planed surfaces; and, if friction rollers are used for plate girders, the ends should be supported on pin shoes in order to distribute the pressure over the rollers so as to make them effective. He would also limit the size of rollers to 3 ins. in diameter.

While the speaker is of the opinion that specifications should not restrict the designer in using his judgment in the selection of details,

Mr. Schneider. he believes, however, that they should confine him to good practice and substantial designs, but prohibit gimcrack construction. The speaker refers more particularly to those antiquated details, which were in vogue when bridge building in the United States was in its infancy, such as loop rods with bent eyes, wing plates, U-nuts, stirrups, loose hangers and other unsubstantial and complicated contrivances. The speaker is constrained to admit that this kind of flimsy construction is practiced by a few engineers, even at the present day, thus showing the necessity for prohibitory clauses in specifications. The few who still cling to the old practice are either those who copy their designs from books, or those who have for years been making routine designs for a bridge shop, and have therefore become so accustomed to work in the same rut in which they commenced years ago, that they are loath to depart from their old ways.

The following clauses inserted in the specifications would prevent this kind of construction:

"Adjustable members in any parts of structures shall preferably be avoided.

"All lateral and sway bracing shall be made of shapes which can resist tension as well as compression.

"All floor beams in through-bridges shall be riveted between the posts, above or below the pin."

The ideal modern structure has few or no adjustable members, and when once erected needs little attention excepting such as is necessary to protect the metal from corrosion.

Another practice which is not consistent with good design, and which is still persistently adhered to by a few designers, is the lop-sided construction, which the author's specifications do not seem to exclude. By lop-sided construction the speaker refers to members of pin-connected trusses with unsymmetrical sections, viz., sections in which the neutral axis is not in the center line of the member. Placing the pin in the neutral axis of an unsymmetrical section by no means distributes the strain uniformly over the entire section, but produces bending strains in addition to the direct compression. It is also unsightly and unconstructive. In order to prohibit this construction, the speaker would suggest the following clause:

"All main members in pin-connected trusses should have symmetrical sections, and the pins should be placed in the line of the neutral axis."

Most modern specifications prohibit the use of cast iron, and where it is necessary to use castings, cast steel has taken its place. The specifications for the quality of material should therefore include cast steel, which is not mentioned by the author. Under "Workmanship," the author specifies: "All workmanship shall be first class," but omits some particulars which are essential to first-class workmanship. The specifications say: "Where the work is to be field-riveted,

the parts shall be assembled before leaving the shops, and the holes reamed to match." This is practicable in only a few cases. The majority of field holes are in the connections of the floor system. The assembling and reaming of these holes in the shop is not only impracticable, but it is bad practice. As all members which are alike should be interchangeable, the field connection holes between floor beams and stringers and between floor beams and posts should be drilled or reamed to iron templates.

The author specifies that: "All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned," etc. Why except flanges of plate girders? The speaker is of the opinion that abutting surfaces in flanges of plate girders should be planed in order to make a good fit and give the work a neat and finished appearance, more particularly as this is the usual practice in first-class bridge shops. First-class workmanship requires also the ends of floor beams and stringers to be faced.

The author limits the play between pin and pin hole to $\frac{1}{16}$ in., but when bolts are used instead of pins a variation of $\frac{1}{16}$ in. is allowed. Why this distinction? If a bolt is used in place of a pin, it should have the same fit; and, if it is used in place of a rivet, it should be turned to a driving fit.

The speaker would suggest the following additional clauses under the heading of workmanship:

"All holes for field rivets, excepting those in connections of lateral and sway bracing, shall be accurately drilled to an iron template, or reamed while the connecting parts are temporarily put together.

"The ends of floor girders shall be faced true and square.

"All eye-bars or pieces which have been partly heated shall be annealed."

GEORGE S. MORISON, Past-President, Am. Soc. C. E.—The speaker Mr. Morison. believes the general practice which has been recommended in this paper, that the live load shall be computed as having twice the effect of the dead load, is one which has been generally recognized as good, for a considerable time. It is the one which he has followed. But whether it should be applied to all classes of strain seems exceedingly doubtful. All the formulas and all the experiments which lead to any such conclusions are based on the rupture of the metal; that is, they are based on the final destruction of the metal by tension. Now, supposing a member never receives anything but compression, it certainly, under that condition, is never going to break by tension. And yet every compression member that anyone has ever seen tested to rupture has broken by tension. This means that when a long compression member breaks, it is enduring strains which are very different from the strains to which it ought to be subjected when in actual work. If the strains in a compression member are kept down to such a limit that nothing but compression exists in that member, under the

Mr. Morison. working strains, there may be no reason for making any difference in the limit of strain allowed for dead and for live loads. There are two ways in which tension appears in a compression member. One is by the member being so long that, through various irregularities, it is deflected out of shape. The other is by the application of the strain outside of the axis of that member. Both of these conditions should be avoided in a well-designed bridge. It is possible to determine, by a careful examination with micrometers, whether the members of a bridge are straight; nothing but compression can exist in compression members so long as the members are found to be straight. In a well-designed structure, if the length of the posts is limited to thirty times the least transverse dimension, the chance of getting tension in the posts, under any working strains, is so small that there is no reason for making any difference in the strain permitted for dead and for live load. It has been the speaker's practice for a good many years to take particular pains to make the compression members balanced sections. In the vertical posts of an articulated pin-connected truss this can always be done. In posts there is danger of tension arising under various disturbing causes, especially as posts are liable, in case of derailment, or otherwise, to side blows. In the chords of an articulated truss it is also possible to balance the section so as to have practically the same resistance above and below the center line. The chords of a bridge are not liable to blows, and the length of chord sections can be kept down to something like fifteen times the least transverse dimension. These are points on which there is a great deal to be said, but it has been the speaker's practice for years, though making exactly the relative allowance that the writer does for the difference between dead and live load in tension, to allow a uniform strain in compression; but while doing so he makes rigid rules, on the lines which have just been referred to, in limiting the length of compression members.

The next matter is the load specified. This paper follows the usual practice of providing two locomotives followed by a certain train-load. Curious results have sometimes occurred in specifications drawn on this basis. A specification was sent out by a prominent western road some years ago in which a locomotive was carefully given, with length between wheels, and weights, followed by a certain train-load, and the train-load was several hundred pounds per lineal foot more than the locomotive averaged. The fact must be faced that there are likely to be very great changes in motive power at any time. There have been such changes. There will continue to be such changes. It is not at all improbable that in many localities trains will be operated by electric power, and trains will be practically of uniform weight throughout, with concentrated weights on wheel bases at various points in the train. In this connection the speaker will present, from a paper pre-

pared for his own private use, the conclusions which he drew some Mr. Morison, years ago as to the proper basis for live loads; this being intended not as a basis adapted to any rolling stock actually in existence, but to the probabilities which must be faced, these conclusions being:

"*First.*—It is inexpedient to proportion structures for any specific form of locomotive, as all such forms are liable to rapid change.

"*Second.*—The present indications are that the train-load per foot is likely to be as heavy as the locomotive load per foot, though on a maximum train-load the distribution is uniform; whereas, it is not so with a locomotive.

"*Third.*—The load on the driving wheel base of a locomotive is liable to be twice as much per lineal foot as the train-load or the average weight of the locomotive per lineal foot.

"*Fourth.*—The driving wheel base of a locomotive may be taken as equivalent to 20 ft., a uniform load extending over 20 ft. being equivalent to an equal aggregate load concentrated on three driving wheel axles 6 ft. 8 ins., between centers, or on four driving wheel axles 5 ft. between centers, or on five driving wheel axles 4 ft. between centers.

"*Fifth.*—On fast passenger locomotives extreme weights are liable to be concentrated on a single pair of driving wheels, these weights being increased by an injudicious system of counter-balancing; to meet this condition one-half the entire driving wheel load should be considered as concentrated on a single axle.

"*Sixth.*—The usual length of the heaviest class of locomotives, as coupled up in the train, is about 60 ft.

"*Seventh.*—The loads may be considered as practically uniform for all lengths exceeding the length of two locomotives (120 ft.); for lesser lengths there should be a provision made for an increased load corresponding to irregular loads and such local disturbances as are caused by flat wheels and other imperfections.

"*Eighth.*—Provision should be made for the rapidity with which strains are put on the web members, this provision increasing from the ends toward the center on a basis which provides an excessive load for those portions of the strains which are greater when the bridge is partially loaded than when fully loaded.

"*Ninth.*—It is expedient to adopt different classes of weights or loads for structures which carry different classes of traffic, but in the interest of simplicity and uniformity it is advisable to observe the same rates of variation between train-load, wheel-base load, etc."

The speaker has followed this practice for years, and it has given very satisfactory results, and wherever he has had occasion to compare it with some special loading which has come out, he has been surprised to see how closely it has agreed with that load. A prominent instance was the case of the special trucks on which the Krupp gun was taken to Chicago during the World's Fair.

The next point to which he would refer is the design of plate girders; and he thinks that the specifications call for a class of work which is not the best. The nearer a plate girder can be made to an absolutely solid piece of metal of its dimensions, the more perfect it is. The fewer pieces a plate girder can be made of, the more nearly it will approach this condition. A plate girder made simply of a web plate and four angles is to be preferred to one with cover plates above

Mr. Morison. and below. There are advantages in cover plates, but every particle of strain that goes into a cover plate must be transferred from the web through two sets of rivets instead of one. In the case of plate girders, like some of the older ones and like some that unfortunately are still made, in which a large number of cover plates are piled up on the two flanges, top and bottom, the strains in the outer of which plates must be transferred by rivets of a length several times their diameter, there are serious doubts whether these outer plates really do much work. According to the theory on which plate girders are proportioned, the farther the metal is from the neutral axis, the greater the amount of work it does. But when the work can only be performed through strains transmitted by long, slender rivets, which rivets can bend, no matter how perfect the work is, the speaker believes that the amount of work done by the cover plates, instead of increasing with the distance from the neutral axis, decreases with that distance. Everything which encourages the use of thick webs in plate girders and a small number of parts in the flanges is to be approved. The speaker, therefore, always proportions plate girders by the moment of inertia, taking into consideration the whole of the web to resist the bending strains; of course, this means the whole of the web as it contributes toward the moment of inertia. Eliminating the web encourages the use of thin webs, and there are many objections to thin webs. A thin web gets out of shape. It deteriorates more rapidly if it rusts. It is much more easily injured in case of any kind of accident.

Another feature in the specification for plate girders which seems to require some change is the matter of stiffeners. The speaker does not believe that it is right to consider a plate girder as an articulated structure. The web must be treated in the same way as the web of an I-beam. It is a continuous web, and the only function of the stiffeners is to act as stiffeners to prevent the web from buckling. There is no established good rule for determining the position of stiffeners; but the deeper the girder is, the closer the stiffeners should be together, if they are to act solely as stiffeners. The following rule is suggested, which is slightly varied from the one the speaker has usually followed: Where the depth of the girder exceeds 50 times the thickness of the web-plate, vertical stiffeners shall be used. The stiffeners shall be spaced at such distances that the product of the distance between the stiffeners by the depth of the girder shall not be more than 10 000 times the thickness of the web. That is, if the depth of the girder was 100 times the thickness of the plate, the difference between the stiffeners would be the same as the depth of the girder.

There is one other point. This specification contains the old provision that the rollers shall be proportioned according to the square root of their diameters. This is entirely wrong. Furthermore, any provision like the one contained in these specifications that the rollers

shall be so enclosed as to exclude dust is simply shutting in what you Mr. Morison do not see. Some years ago the speaker took great pains with a roller bearing; it had a planed steel plate at top and bottom, and the ends of the rollers were enclosed by rubber gaskets under brass plates. It looked all right. When it was examined the rubber gaskets had served admirably to keep the dust in. There is no way of excluding dust which does any good. The only satisfactory method is to make a construction in which the dust will blow out a little more easily than it will blow in; and this can be accomplished by placing the rollers on an open bed, made, as the speaker prefers, of rails placed about $\frac{1}{2}$ in. apart and planed, in which there is free chance for the wind to blow in all directions, and in which, when the dust blows through the cracks, it falls into a space a good deal bigger than the crack, from which the wind will clear it out entirely. The speaker has never had any trouble with dust collecting around rollers since he has used this detail.

But to speak of the pressure on rollers, he thinks that the correct conclusion is briefly stated in these words, which were written five years ago:

"The American practice has generally been to make the permissible weight per lineal inch of the roller proportional to the square root of the diameter of the roller. A common provision in specifications limits the weight in pounds to $500 \sqrt{d}$, d being the diameter of the roller. This rule is based on the theory that there is a permissible limit of indentation, which the roller may be allowed to make in the rolling surface, which permissible indentation is supposed not to exceed the elastic limit. If the versed sine of the indentation is constant, the chord of the same indentation will vary as the square root of the radius of the circle; and as the permissible weight is supposed to be a constant of the chord of indentation, which chord varies as the square root of the radius or diameter, the strain is determined accordingly.

"While mathematically correct, this rule seems to me based on an incorrect conception of the function of rollers. There is no special reason why the strains, either on the surface of the rollers or on the bearing surface, should be kept within the elastic limit; on the other hand, it would be better to have them exceed the elastic limit, if the result of such excess was to reduce the surfaces, both of the rollers and the bearing plates, to the condition of the surface of cold-rolled iron. The only necessary limit is that the elastic limit must not be so greatly exceeded that the metal under the surface will flow, and the roller become permanently deformed, as has occurred in conical roller bearings under turn-table centers.

"On the other hand, the same absolute indentation which would lock a small roller so that it would lie stationary in a groove might do little harm to a large roller. If we assume the roller to be set in a groove, we have forces applied on a bent lever tending to lift that roller out of the groove, the vertical arm of the bent lever being the radius of the roller, and the horizontal arm half the width of the groove. So long as the width and depth of the groove are kept proportionate to the radius or diameter of the roller, the force required

Mr. Morison. to roll the roller out of the groove will be constant. In other words, the formula which will maintain for all sizes of rollers a constant freedom of motion would make the strains on those rollers proportional to their diameters.

"In Germany this rule has been followed, and while there has been a considerable variation in the unit strain allowed, the fact has been accepted that the weight placed on a roller should vary directly as the diameter of the roller.

"The American rule is not only incorrect in principle, but vicious in its results. It encourages small instead of large rollers; a 12-in. roller, for example, being allowed to carry only twice the weight that would be put on a 3-in. roller, so that the smaller the diameter of the rollers, the greater the weight which would be permitted on the same area of bearing surface.

"In proportioning expansion rollers, I have adopted a somewhat arbitrary basis, accepting $500 \sqrt{d}$ as correct for a 4-in. roller, thus making the weight permissible per lineal inch 1 000 lbs. for a roller 4 ins. in diameter. Instead of varying the weight with the square root, I vary it with the diameter; this makes the permissible weight 250 lbs. per lineal inch on a 1-in. roller, and 3 000 lbs. per lineal inch on a 12-in. roller."

CORRESPONDENCE.

J. B. JOHNSON, M. Am. Soc. C. E.—The writer quite agrees with the Mr. Johnson. author in interpreting all fatigue experiments as indicating that live loads require twice as great a factor of safety as dead loads. Indeed, in his "Materials of Construction" (1897), the writer undertook to show (Chap. XXVII), that the formula,

$$p = \frac{a}{1 - \frac{\text{Min. stress.}}{2 \text{ Max. stress.}}}$$

was a true interpretation of Wöhler's results, where p = working maximum stress for combined loads, and a = working stress for live loads.

It is there shown, also, that this formula applies equally for stresses of the same and of opposite signs, and, furthermore, that it is neither more nor less than the old rule of twice as great a factor of safety for live as for dead loads. As this last is the burden of the paper, the writer can but agree with the author. The above formula replaces both the Launhardt and the Weyrauch formulas, as well as those which have long been in use by the Pennsylvania Railroad Company and others. The argument in its favor and the explanation of its derivation are given in full in the work cited, and they will not be repeated here, the significant thing being that the so-called "new method of dimensioning" is, when properly understood, simply the old rule long used by engineers, and given by Rankine, to use twice as great a working stress for dead loads as for live loads. When this is understood clearly a formula is hardly necessary, and the whole theory of a proper interpretation of fatigue experiments loses its interest.

In the chapter cited a new theory as to the real character of the so-called "fatigue" is also advanced. It is the opinion of the writer that this should really be called the "gradual fracture of metals," instead of "fatigue of metals," since the metal away from the plane of rupture never seems to have been changed or injured in any way. His theory is that some one or more of the many small faults in the metal, sometimes called "micro-flaws," are the starting points for cracks, which gradually extend under the repeated elastic deformations until complete rupture occurs. If this is the case, it should not be expected that any great uniformity of results would be obtained on different specimens cut from the same bar, and this is found to be the case.

It is true that proper working stresses would never start the extension of these micro-flaws for any number of repetitions, but if the danger line can be fairly established, then engineers should keep well within it by the use of a suitable factor of safety; and certainly this danger line is very much lower for repeated or for reversed loads than for static loads. In fact, under static loads, these flaws would proba-

Mr. Johnson. bly never gradually extend under any load, however great. The difference between live and dead loads is, therefore, a vital one, however the theory of failure is regarded, and this distinction should certainly be introduced into the proportioning of all metal structures in some rational manner. Apparently no better rule can be found than the simple one of allowing twice as great working stresses for dead as for live loads.

Mr. Schaub. J. W. SCHAUB, M. Am. Soc. C. E.—Aside from the Launhardt formula, which the writer considers more clever than useful, there is no question but that it is rational to assume a different factor of safety for live load than for dead load, in proportioning the sectional areas required in bridge members. This matter was discussed* by the writer, in 1887, when he suggested in lieu of the Launhardt formula that a factor of safety of 3 be used for dead load and a factor of safety of 6 for live load. This suggestion has been very generally adopted by bridge engineers, and notably by Theodore Cooper, M. Am. Soc. C. E., in his "Specifications for Railway and Highway Bridges." If the author will refer to the Michigan Central Railroad specifications for 1896 he will find all his suggestions fully anticipated, including even the variable allowances for impact, excepting that no percentage of increase is added to the live load for spans exceeding 100 ft.

The use of different factors of safety for dead and live loads was first presented to the writer by Mr. Frank D. Moore about 1882, when in the employ of the late C. Shaler Smith, M. Am. Soc. C. E. At that time Mr. Moore used different factors of safety for dead and live-load stresses in chords of swing bridges, near the center, where the dead-load stress is so largely predominant when the case of the span swinging, without end supports, is combined with the live-load stresses.

Referring to the specifications proposed by the author, it might be suggested that in place of the concentrated loads a uniform load be used, combined with a single concentration so placed as to give maximum stresses. As to what this load shall be the writer is not prepared to say, inasmuch as there are a great many views on the subject.

Mr. Wright. CHARLES H. WRIGHT, M. Am. Soc. C. E.—The engineers employed in the offices of the several bridge companies would welcome more cordially than any other class the advent of a specification which would be at once acceptable to the engineers of the railroad companies, and to the bridge builders. These men also realize more keenly than others, perhaps, the apparent hopelessness of such a result being reached at this time.

The editor of a leading engineering journal once said: "No class of men are so intolerant of the ideas of their fellows as the bridge en-

* Transactions, Am. Soc. C. E., vol. xvii, p. 169.

gineers." This is in a measure true. The statement might be applied, Mr. Wright, however, with equal or greater force to other branches of engineering. The more progressive of the ship-builders frankly admit that the bridge-makers are far in advance of them in the design and manufacture of steel structures. If this be true, it must also be true that the bridge engineers, while insisting upon infusing something of their own individuality into their work, have been willing to accept, to some extent at least, the best from the work of others, and to engraft it in their own designs. As the men in the offices of the bridge companies recall (as they must all be able to do) how a set of designs will be heartily approved by one engineer, and be severed limb from limb by the next to whom they are submitted, they must still feel, however, that the time is not yet ripe for a universal specification. Notwithstanding the progress made in bridge engineering, there is still a long transition period ahead of it.

Only a few months ago the bridge companies were anxiously considering how best to meet the requirement of drilled medium steel. Expensive machines for doing this work will soon be idle, if, as seems likely, the present tendency toward punched, unreamed, soft steels continues.

The most a specification like that proposed can accomplish, at present, is to eliminate the meaningless requirements, and more or less absurd clauses which punctuate the majority of the great number of specifications now in use.

Why should an arbitrary assumption of a factor of safety of 2, 3 or 4 be made, or imaginary typical (?) engine concentrations be used, and the stresses then be figured with such minuteness that, if the water bucket happens to be at the wrong end of the engine tender, all the computations are false?

Why should it be necessary in determining pin moments to use a specification which makes it necessary to find new allowed unit stresses at each point where a bar, or other member, rests on the pin?

Why should it be required that hours be spent in figuring dead-load deflections, and changes due to camber, for the sole purpose of making elaborate diagrams showing the notching (even to eighths and sixteenths of an inch) required in framing the ties, only to have it develop later that the bridge carpenter has found it necessary to take out a hump, or sag, of $\frac{1}{2}$ in., or more, sighting with his eye only?

There is no part of bridge anatomy which has not been made the hobby of this, or that, engineer. With one the trough, or hat shape, is the only type of chord to use in a lattice girder. With the next the single web or T-shape is in every way preferable.

Of two engineers using Cooper's specification, one will require that the unit stresses in details be increased for dead-load strains, and the other will feel that he is being imposed upon if this be done.

Mr Wright. The above examples of what is daily going on are given, not as criticisms, but merely to help the statement that the writer does not believe a general vote in favor of any one specification is to be expected at this time.

The points in the proposed specification which the writer would criticise are:

(1) Limit to 3 ins. the minimum diameter of rollers for spans over 150 ft. in length; and, by preference, segmental rollers shall be used. Increase the allowed bearing per lineal inch to $800 \sqrt{d}$.

(2) Omit the clause prohibiting the use of plate girders without cover plates.

(3) There is no necessity for the requirement that built chords shall be spliced for the full area of the section, at milled ends.

(4) Pin-plates should be so arranged as to properly transfer to the pins the percentage of stress borne by the cover plate and by the angles, secondary angles being used if necessary.

(5) Make the length of the plates on built sections one and one-quarter times the greatest width of the member where they come at pin-ends and one-half this length at other points.

(6) Specify the size of lattice to be used on larger sections, as where angle lattice shall be used, and where double lattice shall replace single bars.

(7) There is no reason why shoe and bed plates on small spans should be milled. The surface left by the rolls is less susceptible to the action of rust than the milled surface, and is also fully as smooth as the surface left by the average bridge milling machine.

(8) Requirements, such as: "Rivets shall completely fill the holes," "Rivets shall be countersunk where necessary," "Finished members shall be free from twists," etc., "Eye-bars shall be straight," "Eye-bars shall be bored at the same temperature," etc., mean nothing, and merely add to the length of the specification. The first sentence under workmanship, "All work shall be first-class," would seem to cover it all. Of 90% of the so-called "shop instructions," of the average specification, brief notes on the drawings are all that the shop men read or even see.

(9) The limits of phosphorus, sulphur, and manganese are needlessly severe.

(10) A few specifications require (theoretically, at least) that all material shall be oiled at the rolling mill. A few inspectors have also earnestly tried to have this done. They have generally failed, the failure being largely due to the opposition of the mill men; particularly at those mills where material is often loaded on the cars while yet too hot to oil or paint. It is true, however, that the mill men quietly submit to requirements costing more, and of not one-tenth the value that such a one would be.

That material should be shipped and lie for weeks under all sorts of weather without protection of any kind, is inexcusable. Most engineers, however, treat the matter with indifference. Perhaps they rely upon the statement that the paint "holds better" if the surface be first thoroughly coated with rust.

The result of the writer's observation is that soft steel rusts much more rapidly than iron, when exposed without protection to the weather. He has assembled a set of specimens taken from angles, channels and flats which have laid exposed to the weather for periods of from two to sixteen years. Some of the specimens were oiled soon after rolling, others a year or more afterward, and some not at all. Among the unoiled specimens the iron ones show almost uniformly less rust than the steel ones. A very casual examination is sufficient to show the importance of keeping iron and steel structures thoroughly painted, and to show also very conclusively that the time to apply the protective coating is before any opportunity has been given for rust to begin its work.

J. R. WORCESTER, M. Am. Soc. C. E.—The author's clear and able presentation of the arguments in favor of the use of the Launhardt formula, or some similar one, for proportioning parts, seems to lack one vital link in the chain of reasoning. If, in designing bridges, the engineer were endeavoring to make them fail under a certain load, or at a certain time, or to make all parts fail at the same time when the opportunity occurred, then he could not do better, probably, than to design them in accordance with the knowledge gained from Wöhler's experiments. When he is endeavoring, however, to so proportion the work that it will never fail, or be distorted beyond the "limit of uniform elongation," he is dealing with an entirely different element in the strength of the material.

This "limit of uniform elongation," or "elastic limit," is the real limit of strength of a structure. Beyond this, engineers hope never to go, and a bridge which has reached it is hopelessly beyond its period of usefulness. It is to keep away from this limit that the "factor of safety" is used, and all parts should be designed with a view to being proportionally distant from this point. So far as the writer is aware, neither Wöhler's experiments nor any others have shown that an infinite number of repetitions of a strain less than the elastic limit of the material (strains in opposite directions being added) has the slightest effect upon the elastic limit; and until it is shown, or reasons are given why it should be inferred, that this elastic limit is reduced by fatigue, it does not appear to be proved that in proportioning parts, the engineer should take account of changes in the amount of stress.

There is, however, a logical reason for allowing a greater strain from dead than from live loads, and that is, that, while the former may be exactly known and never exceeded, the latter involves elements of un-

Mr. Worcester. certainty. Not only is the additional effect of impact uncertain, but the amount of the live load itself—especially in railroad bridges—is also very doubtful, when it is considered that a future period of indefinite extent is being provided for.

Acknowledging, therefore, the wisdom of allowing lesser strains from live than from dead loads, the author's method of proportioning parts has many advantages, above all, the consistency with which it applies to details as well as to main members, but its simplicity is largely decreased by the introduction of an arbitrary and somewhat uncertain addition for impact. As an instance of the uncertainty of this addition, take the case of a floor beam of a double-track bridge: Should the same allowance be made for impact, under the specification, as if only one track were carried by the beam? If impact is to be provided for, it is questionable whether the whole addition to the live load cannot be more conveniently and quite as thoroughly provided for in this way, as Mr. C. C. Schneider has done for many years in his specifications.

On the whole, the specifications proposed by the author are very exhaustive and satisfactory. There are, however, some points which may, perhaps, be open to question.

The writer would strike out the clause which reads: "A type of truss shall be used * * * which subjects no member to alternate strains." This would apparently entirely bar out riveted Warren trusses of any kind, which in certain places are exceedingly valuable, if not indispensable.

The author has followed the almost universal practice of requiring transverse bracing at every main panel point of deck bridges. In single-track bridges this requirement may be justified on the ground that it does no harm, although it is not in conformity with the principle of so distributing the metal that the strains in each piece can be accurately calculated, for it introduces indeterminateness in the strains which can be dealt with only by making arbitrary assumptions. In the case of double-track deck bridges, however, intermediate transverse bracing is worse than useless, for the reason that when a single track is loaded and one truss deflects more than the other, a rigid system of intermediate transverse bracing will prevent the trusses from deflecting in a vertical plane, and will throw the track to one side, at the same time making strains in the lateral bracing from purely vertical loads. This will produce an exceedingly unpleasant, if not dangerous, side lurch in a rapidly moving train, and it is often the cause of a continual stretch of the lower lateral rods. Dispensing with the intermediate transverse frames does no harm, for the top laterals and end cross-bracing can easily be made to resist all the lateral force, and the elasticity of the posts to which floor beams are riveted is always sufficient to allow for the unequal deflection of the trusses without disturbing

the connections of the floor. In through bridges this condition is bound to exist, and why not in deck bridges?

The author has followed the common practice of not considering any part of the web of a plate girder as acting with the flange. The common justification of this is not that the web does not actually act with the flange, but that it is often spliced, and consequently inoperative, and that the metal thus introduced does no harm. The practice is, however, not only needlessly wasteful of material, but often results in making a girder much weaker than it might be made with the same material. If the specification allowed the use of the moment of resistance of continuous webs, providing that joints which are spliced for shear only should not be considered as transferring flange strains, it might prevent the common custom of breaking the webs at the center, or where the flange strain is greatest. It would also favor the use of deep girders where the depth is optional, and the advantage of increased depth in the way of stiffness is of no small consequence. The practice also tends to making open trusses more economical than plate girders, which would otherwise be more desirable.

The prohibition of plate girders made without upper flange plates, though supported by good authority on theoretical grounds, is of doubtful necessity, and often very uneconomical, while it involves much labor in fitting ties to rivet heads, and afterward in renewing the wooden floor.

The specification for web plates is deficient in that it allows $\frac{3}{4}$ -in. webs to be used for very heavy shears—provided that flange angles have two rows of rivets, and the same webs, with no wider spacing of stiffeners, must be used where the shear is decreased to a merely nominal figure. While the science of proportioning webs is in a very primitive state at the present time, it certainly is not common sense that the material in the web should not bear some relation to the shear. Again, it surely cannot be intentional to insist on the use of stiffeners wherever the unsupported depth of webs is thirty times the thickness, or for a $\frac{3}{4}$ -in. web, where the unsupported depth exceeds $11\frac{1}{4}$ ins. The many railroad bridge stringers, without vestige of stiffeners, now in service, which, so far as the writer's observation goes, have never shown any lack of strength from this cause, would bear witness to the absurdity of such a requirement. The writer suggests the following empirical formula as one which will give a spacing about in accordance with the best practice, and at the same time pay some regard to the intensity of the shear:

$$d = t(250 - \sqrt{5.2 s})$$

in which d = clear distance between stiffeners, in inches, t = thickness of web, and s = shear in web plate per square inch. The following note should then be added:

Mr. Worcester. "In case the distance between stiffeners, indicated by this formula, exceeds one and one-quarter times the clear distance between flange angles, the stiffeners may be omitted."

The maximum pitch of rivets in the line of strain may well be greater than twelve times the thickness of the thinnest external plate connected, where the rows are not far apart transversely to the strain, as in the case of flanges, where the angles each have two gauge lines.

The wisdom of not specifying any lower limit for the ultimate strength of rivet steel is not very apparent. Though the tendency of the mills may be to get too high steel, it would seem to be rather a radical departure to agree to accept as low material as can be produced.

The writer is not aware of any reason why contact surfaces should be painted rather than oiled before being put together. Such surfaces cannot rust enough to do any harm if not protected at all, and certainly a coat of oil would prevent even incipient rusting in such a place. Moreover, it is a custom of shop men, when painting those surfaces which are to be immediately covered up, to daub the paint brush once over one of the surfaces, the chances being that not over 50% of the area is ever touched by the paint. Under these circumstances, oil is much superior to paint, from the fact that when the parts are assembled it will crawl, by means of its own peculiar power, while any pigment will tend to retard this covering power.

The specifications do not appear to contain any provision for initial strains in adjustable members, and perhaps this is unnecessary, though the effect of this initial strain on struts and connections should not be neglected. The writer would suggest the following clause, the wording of which is taken bodily from the specification of J. P. Snow, M. Am. Soc. C. E.:

"Adjustable members shall be proportioned for an initial strain of 10 000 lbs. When the calculated external strain in adjustable diagonals is less than 20 000 lbs., the maximum strain on the member shall be taken as 10 000 lbs. plus one-half the calculated strain. When the calculated strain equals or exceeds 20 000 lbs., it shall be used as the maximum, and the initial strain ignored."

In the case of a panel where the two opposing diagonals are the same in sectional area, and the resisting struts of so much greater section that they are inappreciably shortened by the tension in the diagonals, this condition is practically true; because, when an external force produces an added strain to one diagonal and the bars stretch, the opposing diagonal is shortened by just the same amount, and, as the changes in strain are practically proportional to the changes in length, it follows that half the external strain is resisted by the increased strain in one diagonal, while the other half goes to replace the strain formerly existing in the other. As the strain from external force increases, it finally reaches a point at which all the initial strain in the opposing

diagonal is replaced, and after that is done there is no more need of Mr. Worcester, considering anything more than the external forces.

When the diagonals are not equal in section, the adjustable member, being the smaller, will be less affected by external strains than its more rigid opposite; that is, less than half the external strain might perhaps be added to the initial strain, but the error involved in applying the same rule is slight, and always on the safe side.

F. E. TURNEAURE, Assoc. Am. Soc. C. E.—The logical sequence of Mr. Turneure. the main operations involved in the determination of bridge stresses and of cross-sections, as it appears to the writer, is: *First*, the determination of the dead-load stresses; *second*, the determination of static live-load stresses; *third*, the addition to the latter of an amount estimated to cover effects of vibration and impact, thus determining as closely as may be the actual live-load stresses; *fourth*, the combination of dead-load with actual live-load stresses by means of the Launhardt formula, or in a manner such as recommended by the author, for the purpose of making provision for fatigue; and *fifth*, the selection of a working stress for dead-load stress, or equivalent dead-load stress, which shall be sufficiently below the elastic limit to leave a margin for undetermined secondary stresses, imperfections in material and workmanship, corrosion and possibly for occasional increase of the maximum stress by a rare combination of loads.

As the writer desires to discuss impact principally, he will touch on that subject first, although this is not the chief topic of the paper.

Impact and Vibration.—In providing for fatigue, or combining live and dead-load stresses for any purpose, it certainly seems proper to use not the live load stresses as determined statically, but the actual stresses to which the members are subjected, if such stresses can be determined. Unfortunately, the existing data on this point are not very extensive, but enough is known to show that the stresses added by impact and vibration are a very considerable percentage of the static live-load stresses, much greater than those recommended as for impact in the author's specifications. Professor Robinson showed by his experiments* that vibration in ordinary spans, up to 150 ft. or more in length, might be expected to frequently equal 15 or 20% of the static deflection, involving, therefore, a corresponding increase of chord stress. In a series of experiments of considerable extent, recently made by the writer,† Professor Robinson's results have been fully corroborated, and, moreover, it has been found that in still shorter spans this percentage becomes much higher, being at least 40% for plate-girder spans of 25 to 50 ft. in length. Furthermore, this excessive vibration is not of rare occurrence, but takes place so frequently as to

* "Vibration of Bridges," by S. W. Robinson, M. Am. Soc. C. E. *Transactions*, Am. Soc. C. E., Vol. xvi, p. 42.

† *Proceedings*, Am. Soc. C. E., Vol. xxiv, November, 1898, p. 783.

Mr. Turneaure. merit full recognition as an inevitable part of the live-load stress. These additional stresses were found to be due rather to the vibration set up by unbalanced drivers and other causes, than to the suddenness of the application of the live loads, but so long as unbalanced drivers must be used such stresses should be taken into account.

The writer is not prepared to offer an impact formula, but considers that nothing less than 50% should be allowed for girders up to 50-ft. spans, and 20 or 25% for spans from 100 ft. to 200 ft. in length. For the construction of a sliding scale, it is thought that 75% and 25% for the limiting spans given in the impact table of the specifications would be conservative figures. For very long truss spans it is probable that little or no provision need be made in the main members for impact and vibration, as such would, in any case, be small, and furthermore the maximum assumed loading for such long spans occurs very rarely. The velocity of freight trains also, in most situations, would be too small to cause vibration. More data on this subject, are greatly to be desired, but the writer believes that the provision for impact should be liberal, and that it is better to err by making it too large than too small. He is very strongly opposed to the method, followed in some specifications, of entirely ignoring impact as such, and in letting this matter take care of itself by using a fatigue formula which ostensibly deals only with the question of fatigue. Probably in the minds of the writers of such specifications the formula is intended to cover the unknown effect of impact, whatever that may be, leaving for fatigue whatever is left over. It would seem that a much better plan is to keep the subjects of impact and fatigue separate, and to use separate formulas, as is done by the author; for by so doing the subject is kept clear, and it is much easier for designers to use judgment in modifying either formula for special cases, or in changing such formulas from time to time as the knowledge of these questions increases. To use a fatigue formula alone, or to treat the subject as is done in Mr. Cooper's well-known specifications, is, in reality, making a much greater allowance for fatigue, or for a margin of safety, in long spans than in short ones, as the impact and vibration is unquestionably a greater percentage of the live-load stresses in the short spans. This variation seems to the writer to be in the wrong direction.

Fatigue.—The author has proposed a method of making provision for fatigue, which gives results that certainly agree very closely with the experiments, and which would seem to be a very convenient formula for practical use. The writer does not wish to discuss the advantages of this over the Launhardt or other formulas, but is disposed to question the propriety of making provision for fatigue to the extent indicated by the ordinary formula or by the method proposed in the paper. The practical limit, beyond which it is undesirable ever to stress iron or steel, when the stress is always of the same kind, is the

elastic limit, for whenever this is exceeded undesirable deformations Mr. Turneaure. are produced in the structure which cause, among other things, a new distribution of stress. The margin between the elastic limit and the ultimate strength is certainly very desirable in case of emergencies, and it is possible that in poorly designed structures secondary stresses are high enough to cause the total stresses to be occasionally in excess of the elastic limit over very short distances, without causing noticeable distortion of the structures. In a well-designed bridge, however, such secondary stresses should be kept low, and the details made of equal or greater strength than the main members, so that excessive and unlooked-for stresses will rarely occur. If the actual maximum stresses never exceed the elastic limit, the ultimate strength of the member, and therefore the margin for emergencies, remains unchanged, no matter how often the stresses are repeated, and, in case the maximum stresses were fully known, the writer sees no reason why it would not be safe to utilize the full elastic strength of the metal. Furthermore, any slight excess of stress over the elastic limit which might occur would be rare, and the element of recovery of the metal would diminish the necessity for special provision for fatigue. In practice, such full provision for fatigue has in reality seldom or never been made, since impact and vibration have been either entirely overlooked, or have been insufficiently provided for in those specifications using a fatigue formula. Even in the present paper the author lays a double duty on his fatigue formula in certain cases where (page 149) he adopts the ratio of 2 to 1 for live to dead-load stress, and then considers this sufficient to provide also for increased stresses due to vibration and shocks, but not to "sudden application," for which a separate provision is made.

On account of the limited knowledge of local secondary stresses, and of the possibility of the maximum stresses occasionally exceeding the elastic limit in some detail, it is probably desirable to differentiate to some extent between fixed and variable stresses, even though the latter are estimated at their true value. The writer would suggest 30% or 40% as a reasonable addition to the actual live-load stresses for fatigue. It is, in reality, as much as is now usually allowed in short spans. For long spans there seems to be no reason for increasing this, as the details are likely to be better and the distribution of stress more uniform. Whatever provision is deemed desirable, the proper live-load stress to be considered is plainly the static stress plus the increase due to vibration and impact. The frequency with which this maximum live-load stress is likely to occur is perhaps a proper subject for consideration in adopting a coefficient for fatigue. This method would give, for spans 50 ft. in length, an addition, first to the static live-load stresses, of say, 50% to determine the actual, frequently occurring stresses; then a further addition of, say, 40% of 150% for

Mr. Turneure. fatigue, giving a dead-load stress equivalent to $150 + 60 = 210\%$ of the static live-load stress. For longer spans the impact would be less, but the percentage for fatigue would remain the same.

For members subjected to a reversal of stress, the limit which must not be exceeded is the repetition limit as determined by tests, this being less than the elastic limit; and it is certainly desirable to have a greater margin between the working stress for such members and the repetition limit than between the working stress for members subjected to but one kind of stress and the elastic limit. However, it is true here as in the other case, that if this repetition limit is not exceeded, the ultimate strength remains unaffected.

In the proposed specifications the stresses due to the centrifugal force appear to be classed as dead-load stresses. This seems to the writer to be incorrect. For bridges on moderate curves, the speed of 60 ft. per second is low, and the horizontal component of the pressure of the train upon the bridge would seem to be as much a part of the live load as is the vertical component. It would seem also that the percentage to be added to this part of the live-load stress for lateral impact and vibration should be fully as great as for the vertical loads.

Mr. Thomson. T. KENNARD THOMSON, M. Am. Soc. C. E.—Why should more formulas be presented until there are some new and reliable experiments for a basis?

Launhardt's experiments cover fatigue, but any formula for proportioning the members of a bridge should also consider impact, and the only basis for such a formula would be an exhaustive set of experiments extending over several years, and on all kinds of bridges. Any engineer who will devote his time to such experiments will earn the lasting gratitude of his fellows.

The empirical formula proposed by the author is taken partly from Mr. Cooper and partly from Mr. Schneider, both of whose methods give good results, but why should they be mixed? A good many engineers waste their energies splitting hairs when they ought to split their strains in two.

On a short line of elevated viaduct, the speaker once had occasion to calculate the difference in cost, caused by using fiber strains of 8 000 and 16 000 lbs. The strain used was 8 000, but if 16 000 had been used, the total cost of the viaduct would have been decreased by only 9 per cent. The writer has seen many cases where it would have been actually cheaper in first cost to have used more steel, on account of the better prices the shops would have given.

The writer is surprised to see that the author advocates making all bidders submit designs and strain sheets, for, in the first place, it necessitates say from five to ten companies doing a lot of work when only one can secure the contract, and, in the second place, it implies

that the engineers of the railroad are not capable of doing what should Mr. Thomson. be their own work.

Again, if the contractors furnish the designs, they skin everything for fear of losing the work, while if the railroad employs a competent bridge engineer to make plans the work is easily controlled, and if the contract is let by the pound, it does not make so much difference who makes the shop plans.

The author adds one more to the long list of engine diagrams, notwithstanding the fact that it is impossible to get one to cover the ground. For instance, an engineer would not be allowed, generally, to design an elevated railroad to carry the author's engine.

MACE MOULTON, M. Am. Soc. C. E.—While it seems obvious that Mr. Moulton. engineers will probably be unable to arrive at any real agreement as to all the clauses in a bridge specification, yet it is apparently always in order to discuss any suggestions aiming at an approach to uniformity in practice. Taking this view of the matter, the writer desires to offer a few comments on the paper.

There is one statement in the paper which brings out a point which has often struck the writer in examining specifications, and that is: "The specifications have been written from the standpoint of the railroad company rather than from that of the manufacturer." Undoubtedly, this spirit in writing a specification is perfectly proper when carried to sufficient length to insure first-class construction within reasonable limits of economy.

Years ago, when the tools and appliances in use were more crude, and when the shops at which such work was done were merely manufacturing, headed and controlled in most cases by plain men of business and men of possibly a high grade of mechanical knowledge, it was absolutely necessary that the engineers designing work, and making the specifications therefor, should specify the exact requirements to the uttermost detail of manufacture. This procedure, through long years of construction under the exceedingly diverse ideas of engineers, has at length resulted in the equipment of shops with many special tools, and facilities for turning out almost anything which buyers really require, or think they require, to meet the conditions of use of the structures to be built.

As time has progressed, the manufacturers have thought it prudent to so specialize their business as to be in position to meet all the requirements of the engineers of railroad companies. This has required them to employ talented engineers, who have devoted most of their professional efforts to this specialty, to attend to the details of this class of work. These shop engineers and structural specialists have naturally been constantly striving to lessen the expense of production, and, at the same time, in most cases, to elevate the standard of excellence of their product.

Mr. Moulton.

The first-class bridge shops, by this growth toward perfecting their special work, having thoroughly winnowed out the chaff, in the way of odd and unnecessarily complicated details of manufacture, are in a position to make suggestions to the buyers of their product which will in many cases simplify and thus cheapen the product, without in any way injuring the resulting structure.

It was undoubtedly necessary, in the early stages, for specifications to be so written as to force the manufacturers to improve, experiment, specialize and perfect their product. The period has arrived, however, when they know their specialty thoroughly, and the nearer a specification conforms to current shop and mill methods, holding up the standard of excellence properly, and carefully following the work through all stages to eliminate errors and personal equations of the shop, the better and more economical the structures will be. To this end let the railroad companies engage specialists to handle their structural work, under the supervision, of course, of the proper chiefs, but see to it that these specialists are really competent, not only to make out strain sheets and figure out and draw details, but that they have had proper shop training, so that such details will not be expositions of some peculiar ideas of the individuals, but will be up to date and of a character recognized as "good practice," both by the railway companies and the manufacturers.

It is probably within the observation of every bridge shop engineer of experience that many structures have been built by them from drawings and specifications of railroad companies which cost the railroad many per cent. more than necessary owing to want of knowledge on the part of a designer of what a shop could do and what it could not do, except at a greatly increased outlay, and giving a result no better for it than if the structure had been designed with more knowledge or attention to prevalent first-class shop practice. Generally in such cases it has not been policy for the manufacturer to make any comments to the buyer, for obvious reasons. Possibly economy in the salary of the designer, or misplaced confidence, has cost railroad companies many dollars in this way.

The experience of all in the difference between the cost of the steel used in bridges to-day and that of even four or five years ago is a case in point. Before that time experimentation was the order of the day, and, necessarily, engineers who specified work had to push to get the manufacturers to perfect their product. Now, however, they know what they can produce economically, and which will answer the purposes for which it is required perfectly well, and have boldly made known to all what they can and will make at a fair price, and that all fancy requirements must be paid for at an increased figure. Designing engineers, consequently, in justice to their companies or clients, have taken to specifying their steel more in accordance with the manu-

facturers' methods, and are designing their structures to meet the qual- Mr. Moulton.
ities of material so made with much surer knowledge of the uniformity
of their material, and with a resulting economy in the cost of the
structure.

While a specification by a railroad company must state just what
is required and bring out points which particular experience has shown
to be necessary, yet it should be broad enough to allow the special
knowledge of the manufacturing engineers to be available and thus
lead to a saving in cost of work and to getting the best method worked
out for meeting whatever special conditions may obtain.

The writer does not see the force of the author's statement that it
is impracticable to obtain fixed end conditions in columns. Struts
solidly riveted in with rivets enough to take the whole strain would
approach the condition of fixed ends pretty closely.

Regarding the proportioning of details for live and dead-load strains,
the most prevalent practice of proportioning the joints for the com-
bined live and dead-load strain would err on side of safety, and the
writer believes that the present practice in rivet units is based on ex-
periments more nearly approaching live-load than dead-load strains.

As the author truly states, the paint question is very controversial,
but when pure oil is put on as a shop coat, and has several weeks to
dry, the writer has found that the surface produced nearest resembles
that of glass or varnish, and that the difficulties in mixing and apply-
ing a pure red lead coating to such a surface are such that the men
"experienced in this work" must be trained in some other sphere than
this, and be nearer perfection than any bridge men whom it has been
his good fortune to engage, to get results as satisfactory as those
obtained with paints of a lighter body.

The writer cannot believe that it is best to be so rigid as the author
is in his general provision that no member shall be subject to alternate
strains. This would be proper in a pin-connected structure, but in a
riveted structure it would militate against economy of design in many
cases. Possibly the author's meaning is misapprehended, and it should
be noted that he provides a method for proportioning members sub-
ject to alternate strains.

In obtaining net sections of riveted tension members the author's
rule seems rather severe, as in the case of rivets staggered about 45°
or less, it would cause all lines of rivets to be deducted.

The writer cannot see why in steel the zigzag section should be taken
at three-fourths value. In some riveted bridges this method of treat-
ment would make the bottom chords 10 to 15% heavier, without justi-
fication by any experiments which have come to the writer's knowledge.

In specifications for quality of material the author leaves out bend-
ing or drifting tests entirely. Surely these should be provided for, if
only for confirmation of other results.

Mr. Moulton. In specifying about wrought iron a very fine quality is called for. The writer notes no provision for its use in the structures, *i. e.*, no special unit strains for iron. If only used for fillers, why specify iron of so high a grade? Should it not be stated just what parts may be of iron?

The author specifies that threads cut on steel shall be filleted. If this is necessary on soft steel, should there not be a provision against square corners in re-entrant angle cuts, such as occur where the flange of a channel or beam is blocked off?

Why are open turn-buckles barred out?

Regarding inspection of work in shops, the writer thinks it would pay the railroad company to employ a competent inspector, who would have some other functions than that of notifying the manufacturer of defects, when he felt so disposed. According to the author, it seems that the manufacturer would be forced to either ignore the inspector entirely, as having no authority, or be subject to delays for consultation with the engineer during the various stages of the work.

Doubtless the contractor should not be released from responsibility in case of defects discovered after the work leaves the shop, but unless the engineer were at the shop very frequently work which would never be discovered at the site of the structure might go through undiscovered by an inspector of the grade indicated. It would seem to be better to have a first-class inspector to whom should be delegated the authority to pass materials and work, unless it is the meaning of this clause that the engineer shall go to the shop and pass on all the work before shipment.

Mr. Ricketts. PALMER C. RICKETTS, M. Am. Soc. C. E.—The author has presented a very interesting paper, and, as might be inferred from his varied experience in structural design, his general specifications would insure the construction of safe and durable bridges.

It should be remembered, however, in considering the paper, that there are three reasons why the use of formulas, depending upon the fatigue experiments of the German experimenters, in the design of the pieces of ordinary structures, is not necessarily rational. The pieces used by the experimenters were small; were neither full-sized nor built pieces; and the method of loading was different from that to which ordinary structures are subjected. In a given time the number of applications of the load in the experiments greatly exceeded the number to which ordinary structures are subjected. The safe allowed unit stresses in structures subjected to moving loads are much smaller than any of those which were shown by the experiments to injuriously affect the material used.

It is well known that in the design of full-sized pieces it is in some cases irrational to use the constants obtained from experiments on small specimens, even when the method of loading the specimens is

substantially the same as that to which the full-sized pieces will be Mr. Ricketts. subjected. It is well known that rest may remove even an apparent set; and there is no proof that a rest, equal to the length of time between trains on a railroad bridge, does not remove every evidence and effect of strain in the pieces of a structure properly designed without the use of a fatigue formula. Some of Wöhler's experiments show the very great difference in resistance offered by the specimens for a small diminution of the load, even when both loads were comparatively large; very much larger than the safe allowed unit stresses used in the design of ordinary structures.

All these things should be carefully considered in deciding whether fatigue formulas, founded upon any experiments yet made, should be used in ordinary structural design. It is not maintained that such formulas are wholly irrational, but it cannot be shown that they are more rational than the methods of proportioning in common use, which have been developed, without the use of fatigue formulas, from general knowledge of structural materials and from observation of existing structures.

These remarks refer to the design of structures. The use of fatigue formulas is much more rational in the design of quickly reciprocating parts of machines, such as the side bars of locomotives.

C. E. FOWLER, M. Am. Soc. C. E.—The extended use of Laun- Mr. Fowler. hardt's formula would seem to be evidence that most bridge engineers have become convinced that it represents actual conditions more nearly than does any other method of dimensioning. That the form is the correct one for a formula for unit stresses would seem easily proven from a purely theoretical course of reasoning.

Nearly twelve years ago the writer, then employed as bridge engineer on an important line of railroad, had occasion to decide how high the unit stresses should go under actual loadings in use, before many old bridges on the road should be condemned. It was the custom among many engineers of roads in that locality not to call a halt until 20 000 lbs. per square inch on iron had been reached. This seemed to the writer to be very unwise, and a method for determining the allowable unit stresses was accordingly developed, and published* some time later.

Starting with the premise that pure live load is twice as destructive as pure dead load, proof of which may be found in Rankine or Burr, it will be seen that when a bridge member is subjected to all dead load the unit stress may be allowed to nearly equal the elastic limit without danger, but when it is subjected to all live load the unit stress must not exceed one-half the elastic limit of the material; for while actual live loads may never equal theoretically pure live loads, they may, in some cases, approximate them so closely as to leave only

* *Engineering News*, April 6, 1889, p. 315.

Mr. Fowler. a small margin for safety. Letting $\text{Min. } B$ represent the minimum stress on a member and $\text{Max. } B$ the maximum stress on the member, then, when the stress is all caused by dead load $\frac{\text{Min. } B}{\text{Max. } B} = 1$, and when the stress is all caused by live load $\frac{\text{Min. } B}{\text{Max. } B} = 0$.

Letting S represent the maximum allowable unit stress and E the elastic limit of the material, then $\frac{E}{2}$ will represent the superior limit of stress when the load is all live, or, as $\frac{\text{Min. } B}{\text{Max. } B} = 0$, the unit stress allowable will be obtained by multiplying $\frac{E}{2}$ by $1 + \frac{\text{Min. } B}{\text{Max. } B}$ or by unity.

When the load is all dead, then $\frac{E}{2}$ must be multiplied by 2, or, as $\frac{\text{Min. } B}{\text{Max. } B}$ in this case equals unity, by $1 + \frac{\text{Min. } B}{\text{Max. } B}$.

In either case $S = \frac{E}{2} \left(1 + \frac{\text{Min. } B}{\text{Max. } B} \right)$, thus giving a rational method with which to replace a dangerous assumption, and a formula identical in form with Launhardt's. The results from its use were very gratifying, indicating unit stresses ranging from 14 000 to 18 000 lbs. per square inch for iron, where it had been customary to allow 20 000 lbs.

That a formula of this kind can be obtained from a purely theoretical foundation, by ordinary algebraic methods, would seem an additional argument in favor of the retention of Launhardt's formula in bridge specifications. With easily constructed tables and a slide rule to facilitate calculations, it is believed also that this is the simplest of all rational methods of dimensioning.

The method given in the specification for the addition of impact is certainly more rational than the common one of making the span length the criterion as to the percentage. Speaking from an extended experience with specifications using impact additions, the writer, however, cannot believe it to be other than a needless encumbrance to calculations. Perhaps when some of the series of experiments which are under way to determine the actual effect of live load are completed, a method may be devised which will almost exactly represent actual conditions. Until this can be done, it is to be regretted that any further modifications of existing modes should be proposed. No one suffers to any great extent except the estimating departments of bridge companies, where sudden changes from one specification to another, repeated many times during the day, are not only confusing, but absolutely disheartening to the wisher for more uniformity, besides being the feature which adds most to the severe labor of estimating.

The reduction of the commonly accepted value for the coefficient Mr. Fowler. of sliding friction, 20% to 15%, would be advisable when the effect was to decrease the stress, but where the stress would be increased, 20% would more nearly represent actual conditions.

While every engineer may find occasion at times to use rivets in tension, yet, to sanction the practice in a specification is to cause license to become liberty and give unscrupulous designers an excuse for bad details.

With such extended use of concrete for foundations a value should be given for bearing on concrete masonry, and it would seem a small matter to give different values for different kinds of stone.

Under "Details of Design," it is specified that no part of the web shall be counted as flange section. In the large girders now being built for heavy loadings, the number of cover plates to be piled up can be very wisely reduced by carefully designed longitudinal web splices which will develop the strength of one-sixth the web for flange section, and certainly no objection can be found to this with webs of from $\frac{3}{8}$ to $\frac{1}{2}$ inch in thickness.

The provision for fully splicing the flanges of girders is a wise one, as no dependence can be placed on the abutting of plates or angles; for, no matter how carefully they are planed and fitted, they are almost sure to draw apart in riveting.

The method of proportioning tie plates for transferring the stress borne by each segment is the only logical one, and should be adopted in all important specifications.

The requirements as to length of span which shall require rollers have never been correctly stated in any specification. The providing of rollers should not depend alone upon span length, but also upon the weight supported or size of bearing plate, using them when the size of bearing exceeds 200 sq. ins.

Other things being equal, the best designed pin-connected bridge is that in which the eye-bars are packed as nearly parallel to the axis of the bridge as possible, disregarding the requirement to so pack them that the moment on the pin would be a minimum, figuring the pin after the bars were properly packed and using the size so obtained, provided it was reasonable, and only in case it was not, adopting some other arrangement for packing.

Under "Quality of Material," the steel is specified to have an ultimate strength of from 52 000 to 60 000 lbs. per square inch. Not only is this out of the ordinary limits of most mills, and consequently somewhat hard to obtain promptly, but the writer believes that a vote of those interested would show a decided majority in favor of a soft-medium steel, having an ultimate strength of from 55 000 to 65 000 lbs. per square inch.

A feature under the heading "Workmanship," worthy of commen-

Mr. Fowler. dation, is the requirement for the assembling in the shop of adjacent chords and spliced members which are to be field riveted. This will meet with strenuous objection from shop people and will doubtless add to the cost of work; but it will prevent many misfits and much butchery in the field, to the great betterment of the work.

The subject of "Painting" is one about which the last chapter will not be written for many years, and, until then, it behooves the purchaser to see that the material is kept from the weather as much as possible until it shall have received its first coating. This first coating will never adhere perfectly until the mills shall devise some means for removing the scale from steel, or until the bridge shops are provided with facilities for pickling the steel to remove the scale.

While a specification like the author's, with such a large number of commendable features, calls out such an interesting presentation of experience from engineers, it is most sincerely to be hoped that the time is not far distant when the effort will be for reducing, instead of increasing, the number of bridge specifications.

Mr. Snow. J. P. SNOW, M. Am. Soc. C. E.—If proper limits are assumed it makes but little difference in the final results whether the live load is augmented by some multiplier and added to the dead load and a constant unit strain used, or a reasonable impact allowed for, and varying unit strains used. There is undoubtedly a little less work in proportioning the parts of a bridge by the former method than by the latter, but the difference is slight when proper tables and the slide rule are used. In the opinion of the writer, a specification and strain sheet are more intelligible when actual strains plus reasonable impact are shown and varying units used, than when a system like that recommended by the author is used.

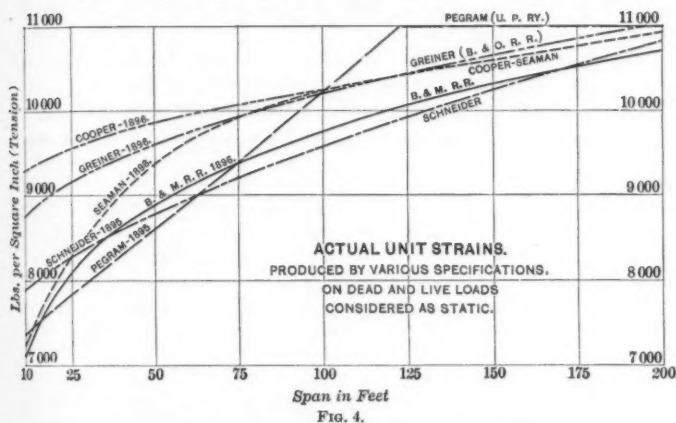
It would seem that the impact produced by unbalanced drivers and defective track and wheels, would better be provided for by a certain actual load applied at any one point in the length of the span. The blow from a flat wheel, for instance, will be the same in actual amount on a 10-ft. span as on one of 100 ft. The practice on the Boston and Maine Railroad is to provide for an impulse at any point equal to a static load of 12 500 lbs. in excess of the typical load, when figuring for consolidation engines; and 15 000 lbs. when using the two-axle load. The different schemes of percentages used by various designers approximate this.

The variation of strains should certainly be applied to all connections fully as much as to the main members.

The Launhardt formula, as modified by William Cain, M. Am. Soc. C. E., $a = u \left(1 + \frac{\text{Min.}}{\text{Max.}} \right)$, is intended to cover both impact and fatigue. If identical coefficients are used with this and the 2 to 1 method, the same results will be obtained at the limits of all live load

and all dead load, but at intermediate points, the two methods vary Mr. Snow. as much as 12 per cent.

Wöhler's experiments with values of $\frac{\text{Min.}}{\text{Max.}}$ between the limits of 0 and 1 seem to be altogether too few to build on to such an extent as has been done. When plotted with values of $\frac{\text{Min.}}{\text{Max.}}$ for abscissas and unit strains for ordinates they fit the Cain formula fully as well as the author's curve; showing a tendency to fall in a straight line or in a curve that is convex upward rather than in a curve convex downward, as is the 2 to 1 formula. The points are so few that it seems ridiculous to contend that any particular formula is established by them. The experiments as a whole undoubtedly prove that the internal



structure of material is changed by strain; that the molecules move one on another, and that if the strain is great enough to make this movement continuous each time the load is applied that the piece will ultimately break. It is doubtful, however, if the actual strains usually occurring in good practice will produce this effect. It is likely that a condition of increased elasticity and decreased ductility will be reached that will totally resist all further molecular change under the strain applied. Probably the jar and tremor produced by a moving load in a member when under heavy strain is more effective in producing this rearrangement of molecules than the absolute range from maximum to minimum in ordinary bridges.

To express the effect of jar on a member by means of a formula is altogether beyond the capacity of science at present, and it seems that a reasonable course to pursue is to adopt a scheme of impact adjust-

Mr Snow. ment or unit strain variation, or both, based on actual observation of various types and sizes of bridges. A scheme that produces unsatisfactory bridges is not right even if it is based on some interpretation of Wöhler's experiments.

Perhaps a fair way to compare the practice of different designers regarding impact and fatigue is to assume certain live and dead loads for different span lengths, apply the various rules prescribed for impact and unit strains, divide the modified loads by the allowable strains, and divide the sum of the assumed dead and live loads by this quotient. The result of this division will be the unit strains produced by the actual load considered as static. The diagram, Fig. 4, shows the results of this calculation as applied to the author's recommendations; Cooper's General Specification, 1896; Greiner's Baltimore and Ohio, 1896; Pegram's Union Pacific, 1895; Schneider's Pencoyd, 1895; and the Boston and Maine, 1896.

The assumed loads were obtained from a diagram of weights of iron bridges and corresponding uniform live loads published by G. H. Pegram, * M. Am. Soc. C. E. Class D was used; the equivalent load per foot being $\frac{60\,000}{\text{span}} + 3\,600$. A track load of 400 lbs. per foot was added to the diagram weight of bridge, and the sum taken as the dead load. For the sake of obtaining continuous curves, the unit strain prescribed for eye-bar tension was used for all spans, without considering the change for shapes prescribed by some for the shorter spans.

If the weight of bridges produced by the individual specifications had been deduced, instead of using the diagram weights for all of them, the unit-strain curves would have come somewhat nearer together, but the relative positions would be very nearly as shown.

The writer's experience would lead him to judge that the author's specification will produce disproportionately heavy bridges of 30 ft. span and less, when compared with those of 75 to 125 ft. It would seem that the impact allowance, while being ample for the shorter spans, should be greater for the longer and should extend to those of greater lengths than is proposed.

The rigidity of a bridge is of fully as much moment as its ultimate strength. In recognition of this principle, and to discourage the temptation to use unreasonably shallow beams and girders, the Boston and Maine specification contains the following clause:

"In girders the flange units shall be modified by the ratio of effective depth to length as follows:

"The tension unit shall be $a \left(\frac{1}{2} + \frac{5d}{l} \right)$, in which d is the strain depth and l the effective length of the girder. But it shall never exceed $1\frac{1}{2}$ times the value of a ."

* *Engineering News*, Vol. xxv, p. 29.

The tension unit corresponding to the ratio of $\frac{\text{Min.}}{\text{Max.}}$ is represented Mr. Snow. by α . The compression unit is similarly modified.

A similar reduction should be used in shallow deck and pony trusses, but an increase for deep trusses would not be advisable, as the chords are light enough without this modification.

A somewhat similar assessment on long, narrow panels of lateral bracing is provided for by the following:

"Tension in laterals shall not exceed 10 000 lbs. per inch; nor $10\,000 \frac{2d^2}{l^2}$, where d is the distance center to center of trusses and l the length of panel."

It is suggested that something to cover these points would be an improvement to the specification under discussion.

Unless riveted trusses are ruled out altogether by the specification, a point about which the writer is uncertain, it would seem that there should be two formulas for struts. The compression members in the webs of riveted trusses are certainly fixed, and the ends must necessarily remain straight and in the line of strain. A pin may tend to hold a member in this way, and it may not. The top chords of pin trusses should certainly not be considered as fixed, and those of a riveted truss are only partially fixed. It is the custom of the writer to consider partially fixed columns as hinged, with lengths reduced to two-thirds or three-fourths the actual length of the member. The Rankine formula allows too much load on very long, slender columns. It is suggested that some limit or qualifying clause should be used to rectify the formula for high values of $\frac{l}{r}$.

There seems to the writer to be no good reason for neglecting the web in computing the moment of resistance of plate girders. The edge of the web plate must take the same unit strain as the flange, and this must decrease toward the neutral axis. If these conditions obtain, the web is resisting moment and must be carrying load. To say that the web is left free to carry the shear by neglecting to use it for flange is nonsense. The facts of mechanics cannot be changed by assumptions. The claim frequently made, that omitting the web in the flange computation is a measure of safety, is caution thrown away. Plate girders need no special favors when compared with other types of structures, and if figured with all the latitude that can be defended, they will still be better bridges than trusses figured to the same units.

It is not apparent why open turnbuckles for adjustable members should be ruled out. The preference of the writer is exactly opposite to the requirement made in the specification. Sleeve nuts have been known to be split from end to end from the freezing of water which filled the space inside on account of the upper end admitting water

Mr. Snow. while the lower end was tight. Open buckles show the condition of the screw ends much better than closed sleeves, and can be kept from rusting much more effectually.

The author uses soft steel throughout. Is there any objection to higher steel if the work on it is such as not to endanger it? The punch and shear are the enemies of high steel in ordinary bridge work. Eye-bars may well be of medium steel, and pins and rollers of still harder material. Hard steel rollers between hard steel or cast-iron plates may well be loaded higher per lineal inch than allowed by the author.

It is certainly well for engineers to discuss frequently the various points of bridge designing, but a standard specification adopted by so high an authority as a Committee of the American Society of Civil Engineers is probably not desirable. Like some other literature extant it might be made to mean far more than was originally intended. Furthermore, the specification does not design the bridge. After innumerable standard specifications shall have been written it will still need a good designer behind the pencil in order to get a good bridge.

Mr. Waddell. J. A. L. WADDELL, M. Am. Soc. C. E.—The author deserves the hearty thanks of the profession for presenting a paper, the discussion of which will result in one more step toward the attainment of that great *desideratum*, standard specifications for bridges. The writer, however, is of the opinion that the revival of the fatigue theory as applied to bridges is a step backward. The paper makes it very clear that such a thing as fatigue of metal really exists, when the metal is strained above what is commonly termed the elastic limit, but shows no good reason for assuming that it will apply in the case of bridges.

The conditions affecting the metal in the tests and those affecting it in bridges are essentially and fundamentally different. In the former the load is applied every few seconds so that the metal has no chance to recover its equilibrium after one application before the next load comes on, the intensity of stress is above the elastic limit, and the number of applications of the load is very great; while in the latter the loads are applied at comparatively long intervals, even in elevated railroads, so that the metal has a chance to recover its equilibrium between loads, the intensity is only about one-third as great and never half as great as that used in the tests, and the total number of applications of the loading is comparatively small. Such being the case, it appears to the writer to be unsound logic to deduce that what holds true for the metal in the tests will also hold true for the metal in the structures.

That fatigue of metals does exist is recognized even by the uneducated. The writer was struck with this a few weeks ago when fishing for tarpon, with piano wire snells which were rather light. After he had just lost a fish by the breaking of a wire, the boatman, an unedu-

cated foreigner, advised that he use two or three snells, one each day, Mr. Waddell, so as to give the wires a rest, and assured him that they would then break less often. The boatman stated that he had found this out from experience.

In respect to reversing stresses, the writer believes that the time element is a most important factor, or that if the condition passes immediately from tension to compression, then to tension again, etc., the effect will be much more injurious than in the case where the metal has a chance to rest between the applications of opposite stresses.

A glance at pages 142, 143 and 144 shows that, for stresses of one kind, the smallest intensity producing rupture is in excess of the elastic limit, and that for stresses of opposite kinds the sum of the two intensities when rupture is produced is also greater than the elastic limit. There is nothing in Wöhler's experiments to warrant the deduction that every live load is twice as destructive as a dead load. The live load of the experiments was probably just twice as effective as a dead load, in that it was applied suddenly, but the live loads on bridges are not applied suddenly; *i. e.*, it takes an appreciable time to bring them on. For the last twenty-five or thirty years, engineering students have been taught the mathematical demonstration that a suddenly applied load produces exactly twice the extension of a bar in tension that the same load applied very gradually does; and the correctness of the theory has been proved by actual experiment. This idea of double effect, therefore, is no new thing; nevertheless, when it is applied to bridge members, it is, in the writer's opinion, absolutely wrong. Does it not seem evident that if it is proper to apply it to a panel suspender, it would be anything but proper to apply it to the bottom chords of a 500-ft. span? This question is one of actual intensities of working stresses, and can be settled finally only by experiment.

If, for instance, it is known that under the worst possible circumstances the metal in any eye-bar of a bridge is strained actually not to exceed, say, 18 000 lbs. for medium steel, it is taken for granted that it can never be worn out by the loading. Why, then, need engineers trouble themselves to make investigations concerning the least as well as the greatest stress, and complicate the proportioning fully 100%, simply to comply with some experiments that "have nothing to do with the case"?

The simplest, most scientific and most satisfactory method is to add a varying percentage to the greatest live-load stress found on the assumption of static loading, the proper percentage being taken from a table. It is true that the exact values of these percentages are as yet somewhat a matter of conjecture, but some day they certainly will not be. In the writer's opinion, there is no more praiseworthy task for the American Society of Civil Engineers to undertake than to in-

Mr. Waddell. vestigate thoroughly, by numerous experiments and adequate apparatus and appliances, the actual intensities of working stresses for all members of all approved types of modern steel bridges. It would take time and money to do this—the former could be given by the members of the Society, and the latter could be obtained from one or more of America's broad-minded and generous millionaires, provided the matter were properly presented to them.

It seems to the writer that the author complicates designing by using impact and the Launhardt formula combined, and, certainly, his proposed impact allowances are not right, as they reduce to zero for spans exceeding 80 ft. in length.

The paper states that:

"The author is not aware of any specification which has made this distinction between live and dead strains in proportioning details, except those which adopt the Launhardt formula throughout."

The writer published the following in 1898.*

"In spite of all that has been said to the contrary in the past, or that may be said in the future, the impact method of proportioning bridges is the only rational and scientifically practical method of designing, even if the amounts of impact assumed be not absolutely correct; for the method carries the effect of impact into every detail and group of rivets, instead of merely affecting the sections of the main members as do the other methods in common use."

The writer endorses most heartily the author's statement that:

"The details are the most important part of a structure, and those experienced in maintenance will testify that it is the live strain which causes loose rivets and which wears out the bridge. To provide for dead and live strains in the main members, while making no such provision for the details, is to build a scientific structure upon a crude foundation."

This principle was really the true *raison d'être* for the writer's book, before mentioned, and will be found to pervade it from beginning to end.

The writer also endorses most heartily another statement by the author, who, in speaking of inspection says: .

"There is no element in bridge construction more important, if properly done, yet none so likely to be neglected. Few appreciate the importance of perfect mechanical work until they maintain the structures they build."

In the specifications proposed by the author it is stated that: "A type of truss shall be used in which the strains may be readily calculated and which subjects no member to alternate strains." Compliance with the latter portion of this specification would involve the abandonment of the Warren girder, which is most useful for deck structures, and rigid diagonals near the middle of truss bridges, and would force designers into using adjustable counters, a type of construction which is rapidly becoming antiquated.

* "De Pontibus," p. 7.

The distance of 12 ft. 6 ins. between centers of railroad tracks is Mr. Waddell too small. Most roads require 13 ft., and some of them more.

The reason for not spreading stringers farther apart between centers than their depth is not apparent to the writer.

The writer believes that an assumed wind pressure of 50 lbs. per square foot on an empty bridge is altogether too high, and is entirely unwarranted. All ordinary single-track railroad bridges with eye-bar bottom chords would double up like jack-knives under such a pressure.

It seems unnecessary to proportion the compression flanges of a plate girder by formula; because, if it be stiffened properly by bracing, the same section as that used for the tension flange will be ample.

In specifying 300 lbs. per square inch as the greatest allowable pressure on masonry, the author would load common masonry too high, and would not load the highest classes as much as would be legitimate. Surely there ought to be some distinction in this particular between American natural cement, concrete or brickwork, and granite of the best quality.

On page 159, what is meant by "unequal tearing"? It appears to be an unfortunate term, as any tearing whatsoever would be likely to soon cause the destruction of the bridge.

The writer believes that the web of a plate girder certainly aids the flanges in resisting bending, and therefore ought to be figured on as so doing. If a designer feels confident that this is the case, he is not likely to put in splice plates proportioned only for shear and inadequate to develop the full strength of the web acting as a beam.

The writer takes exception to the clause of the specifications which compels the use of cover plates from end to end on all plate girders. They are an intolerable nuisance to the trackmen on account of the rivet heads. In elevated railroad work cover plates are barred out for this reason. Can the author show good reason, from practical experience in maintenance, for this requirement?

In respect to the stipulation that "all segments of members in compression, connected by strapping only, shall have terminal cross-bracing plates at each end, the rivets and net section of which shall be sufficient to transfer the total maximum strain borne by the segment," the writer would state that once, years ago, when acting as contractors' engineer, and when endeavoring to make, in the office of the consulting engineer, who had inserted a similar clause in his specifications, a design to satisfy the said engineer, he found that in one span the batten plates would not only meet at the middle of the panel, but would have to lap past each other.

Friction rollers $2\frac{1}{2}$ ins. in diameter are rather small. The writer's limit is 3 ins., and it is not unlikely that this also is too low. Rollers of small diameter are likely to rust and fail to act.

Mr. Waddell. It is all very well to specify in respect to workmanship that: "The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than one-sixteenth of an inch will occur"; but there is no bridge shop in this country which can comply regularly with this requirement.

The writer's specification* is about as stiff as the shops can well bear.

In the writer's discussion of the paper entitled "The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel,"† by E. Herbert Stone, M. Am. Soc. C. E., he made a comparison of the "nominal stresses" involved by using the impact formula and intensities of "De Pontibus" with those given by Mr. Stone, using as a basis the bottom chords of single-track railroad bridges. As a matter of curiosity, the corresponding "nominal stresses" by the author's method and specifications have been computed, and the results have been recorded in the following table, the author's intensities being increased so as to correspond with medium steel by multiplying them by the ratio of ultimate strengths, viz., 65 : 56.

Span, in feet.	Live load.	Impact.	Dead load.	Nominal intensities per "De Pontibus."	Nominal intensities by the author increased for medium steel.	Percentage of difference.
100.....	4 266	2 844	1 630	12 144	12 121	0.2
200.....	4 030	2 303	2 130	13 102	12 630	3.6
300.....	3 860	1 980	2 820	13 966	13 240	5.2
400.....	3 760	1 671	3 500	14 631	13 765	5.9
500.....	3 700	1 480	4 226	15 169	14 243	6.8
600.....	3 660	1 331	4 930	15 585	14 651	6.0

It will be seen that the author and the writer would not be so very far apart in their intensities of working stresses, if they were using the same kind of steel. However, if the comparison were made with web members, especially where the stresses reverse, the divergence would probably be greater.

There is one point not directly connected with the paper, but brought out by a comment upon it in one of the technical papers, which the writer would like to mention, viz., that in his specifications‡ in spite of the use of an impact percentage, there are three tensile intensities adopted. The explanation of this is, that there ought to be a distinction between steel in eye-bars and the same metal in built tension members, on account of the injury to the latter by punching, etc., in the shop; that there is no real relation between

* "De Pontibus," p. 254.

† *Proceedings*, Am. Soc. C. E., May, 1898, p. 364.

‡ "De Pontibus," Chapter xiv.

the tension in an eye-bar and the extreme fiber stress in a rolled or Mr. Waddell, built beam; and that it did not seem practicable to so adjust the impact formula as to make it apply to both chords and suspenders without using two different intensities of working stress. Consequently the specifications give 18 000 lbs. for chord-bars and eye-bars in diagonals; 16 000 lbs. for shapes in same, for eye-bars in hip-verticals and for adjustable truss members; and 14 000 lbs. for flanges of built beams and for shapes in suspenders.

THEODORE COOPER, M. Am. Soc. C. E.—The remarks made by the Mr. Cooper, writer in the discussion of the paper on "The Kentucky and Indiana Bridge,"* are the views he would now express as applicable to the subject under discussion.

The writer believes that if all the various anomalous ways in which retaining walls stand up or fall down in spite of the theory of earth-works pressures, were classified under the term "fatigue of stone walls" it would be generally considered as "absurd and unscientific." The term "fatigue of metals" is equally absurd and unscientific, and has been a stumbling block to the comprehension of the true principles governing the phenomena classed under this term.

In all branches of science the working theories of the day have had to be rejected, modified or broadened as new phenomena, observed or experimental, have shown the deficiencies of the existing theories.

The theory of the limit of elasticity as heretofore held, and which was based upon experiments commencing at zero and extending to some determined maximum of tension or compression, falls to the ground when the tests, as in more modern experiments, take a more miscellaneous range. It does not follow, however, as the author states, that Wöhler's experiments "entirely destroyed the theory of their (the metals') perfect elasticity," but they do show that the theory as to limit of such perfect elasticity is wrong. Wöhler's tests are in fact the most perfect proofs in existence of the perfect elasticity of steel and iron, within certain determined limits. If a piece of metal remains absolutely unaltered after many millions of applications of strain of various kinds and amounts, could any better demonstration of its perfect elasticity be made?

The author appears to have become confused by the various definitions of the term "elastic limit." That it is necessary and proper for laboratory refinements to "split hairs," need not bother the practical workers in any science. Part of the confusion arises from a misunderstanding. Perfect elasticity and uniform elasticity are not necessarily the same thing. The engineer, from a practical point of view, considers a structure to have perfect elasticity, if under any number of repetitions of the loading it remains unchanged, and returns, after rest, to its original form.

* *Transactions, Am. Soc. C. E.*, Vol. xvii, pp. 181-82, October, 1887.

Mr. Cooper. That all test pieces or finished structures may show a slight set has been well recognized from the earliest experiments, and is due to two reasons: *first*, no material is free from internal or initial strains; *second*, the sluggishness of the elasticity; more or less time being required before the piece can return to its original condition.

The writer cannot agree with the author that "this slight set is probably the true explanation of the results obtained by Wöhler."

On the contrary, he thinks that Wöhler's tests show clearly that for practical purposes this small set does not affect the perfect elastic action of the material, when the strains are confined within certain limits.

It is surprising to find the author saying that "many engineers have preferred the concise term, 'Yield Point,' but others still adhere to the old name, 'Elastic Limit,' trusting to a considerate profession to give it the revised interpretation." Does he mean to assert that "yield point" is more concise than "elastic limit," and that those who use the term "elastic limit" are alike confused in regard to these two points? If so the writer desires to go on record in protest. Modern commercial testing is much to blame for this lack of clearness of view. While it does require great refinement of tests to determine with minute accuracy the exact point of the elastic limit, there is no difficulty in determining it near enough for all practical purposes, with ordinary care or ordinary instruments of measurement. The common commercial method of determining it by the drop of the scale beam is absolutely false, and gives errors as high as 10 000 lbs., and always in excess of the true elastic limit.

It is time for those who believe that the "Elastic Limit" is the true test of the capabilities of structural material to point out the falsity of these determinations and insist on the proper methods being used occasionally, as checks upon the cruder and false methods. Any claim made for elastic limits of ordinary structural steel that exceeds 50% of the breaking strength may be looked on with grave suspicion, and should only be accepted after proper methods, carefully worked out, support the claims.

The coincidence, shown in the last column of the table, on page 146, is interesting, and may be more than a mere accident, but the theory formed by the author in explanation is not satisfactory.

If the range of action of the strains is considered, the same figures result, viz.:

First.—The strain passes from zero to — 17 120 lbs., then returns to zero, then to + 17 120, and then back to zero; having passed through a total range of 68 480 lbs.

Second.—The strain passes from zero to — 34 240 and back to zero, or again over a range of 68 480 lbs.

Third.—The strain passes from zero to — 47 080 and back to — 25 680, which also covers a range of 68 480 lbs.

This coincidence does not tend to prove any theory of work, but Mr. Cooper, rather to show the probability that the elastic capabilities of this material are measured by the sum or cycle of strains through which the piece passes, which, in this case, is 68 480 lbs.

ROBERT GILES, M. Am. Soc. C. E.—The writer cannot agree with Mr. Giles, the author that the experiments quoted form a sufficient basis on which to establish a formula for bridge work. In the bridge work under discussion, it is assumed that the metal is not strained up to the elastic limit. There is a factor of safety to keep it well within that limit, and so long as the metal is not strained up to that point, the theory of the fatigue of metals will not apply. The writer agrees with the author, however, in making the dead-load unit strain twice the live load, and is also thoroughly in accord with him in making provision to proportion the riveting according to both dead and live-load strains.

In the specifications, the limit of 5 ft. for the spacing of stringers and deck-plate girders seems to be too narrow, and in standard-gauge track would bring the rails directly over the stringers. A better limit would be 6 ft. 6 ins.

The author, besides rating the live load at twice the dead load, provides for impact, on an arbitrary assumption, and the writer considers that with the proposed unit strains, impact will be sufficiently provided for.

Although providing for certain wind pressures, the author does not state which should be considered as dead or live loads.

In regard to rivets in tension, the author provides that the strains should not exceed one-half of the limit allowed for shear, which may seem a low enough limit; but would it not be better to make no allowance for rivets in tension, and make any details requiring them subject to special approval? Rivets in tension undoubtedly do considerable work, as can be seen in several bridges on the New York Central and Hudson River Railroad, but it would be much better to avoid using them.

The author provides for a single unit stress for the pressure of the rollers. To be logical, this should also be expressed in terms of the dead and live loads.

The minimum limit of 4 ft. between stiffeners in plate girders, specified by the author, would prohibit stiffeners over the bed plates and points of local loadings.

The author's requirements for steel are too low for such pieces as pins, eye-bars, roller bearings and sliding plates. Steel for such members should have an ultimate strength of not less than 64 000 lbs. \pm 4 000 lbs., with an elongation of not less than 25 per cent.

The writer does not understand the clause: "When bolts are used instead of pins, a variation of $\frac{1}{8}$ in. will be allowed between diameter

Mr. Giles. of bolt and hole," when the author has just specified that "no variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{64}$ in."

The writer would suggest that specifications for painting should provide that all painting be done under cover, and that no work be allowed to leave the shops until 48 hours had elapsed from the time of the application of the paint. Bridge work is often loaded on cars while the paint is still green, and the writer has heard of its being painted even on the cars, while they were being hauled out of the bridge company's yard.

In order that there should be satisfactory inspection, and that the railroad company should know what sort of workmanship they are getting, it would seem to be a good thing for all inspectors to report in detail what mill and shop errors are found during the manufacture of the work, and the writer would suggest a clause to that effect.

Mr. Greiner. J. E. GREINER, M. Am. Soc. C. E.—The practice in regard to certain features in bridge design, such as live loads, unit stresses, column formulas and impacts is assuredly in an unsettled condition. This state of affairs is the natural result of a quite prevalent inclination among students, engineers and contractors toward availing themselves of their unquestioned prerogative of compiling specifications whenever they please, and embodying therein such individual fancies as may give the appearance of originality.

No one person, acting alone, can ever hope to bring about any degree of uniformity, so long as this prerogative exists, and so long as the conditions involved are not so much questions of fact, but of opinion, where each individual believes his own *dicta* to be about as good as another's. Engineers will not, therefore, acknowledge any one specification as being so far superior to all others as to be specially worthy of general adoption, and the much-desired uniform practice can be attained, if at all, only through the recommendations of a body of representative engineers who are willing to waive their own individuality for the sake of uniformity. It is now time that a committee of such men should be appointed by this Society, men who have the full confidence of both engineers and contractors, because the existing specifications are of such a variety as to indicate to the ordinary layman an unsettled condition which, to say the least, is not creditable.

An engineer, when compiling specifications, should not lose sight of the fact that two interests, namely, those of the railroad company and those of the manufacturer, are so closely blended that good and economical structures are not likely to be obtained unless both sides are considered. If written from the standpoint of the engineer only, a good bridge may result, but at an excessive cost. If written from

the standpoint of the manufacturer, cheap shop work will probably be the primary consideration and the bridge may lack all other qualities. Substantial and economical structures are demanded by the railroad company, and every specification should aim to embody both of these features. The author states that his specifications are written from the standpoint of the railroad company.

A specification should be explicit. It should not leave to the decision of the contractor such important details as floor beam and stringer connections, the least dimensions of posts and bracing, and the limiting ratio of $\frac{l}{r}$. There are many ways of connecting floor-

beams. They can be riveted to posts entirely below the chord pins, or above the chord pins with lower flange above the heads of eye-bars or with lower flange below the heads of eye-bars and on the line with the bottom of posts and connected thereto by means of plates which will furnish a connection for lower laterals. Floor beams also may be suspended by single or by double yoke hangers, by eye-bolts or by plates. In deck bridges they may rest upon the upper chords, or be riveted to posts below the chords. Stringers may rest upon the upper flanges of floor beams, may be riveted to the webs of floor beams, or be both riveted to webs and rest upon substantial brackets. Since all sizes and thicknesses of angles are rolled, the contractor should not be the one to decide whether end-connection angles of stringers and floor beams should be $\frac{3}{4}$ or $\frac{5}{8}$ in. It has been practically demonstrated that some of the above methods are better than others, but the author having given no preference for the best will, therefore, get the cheapest.

The light lateral bracing used a decade ago in spans ranging from 20 to 150 ft. can hardly be considered good and effective for bridges designed for a 50% increase in live load. This bracing is not put into a bridge merely for the purpose of sustaining a wind pressure of 30 lbs. per square foot; in fact, a through-truss bridge would collapse under a light live load in a perfect calm, if the lateral bracing should be taken out. The primary object of lateral bracing is, therefore, to tie the structure together, to resist deformation under the action of the live load, and to eliminate lateral vibration as much as possible. In order to obtain these results satisfactorily, lateral bracing should bear some relation to the parts which it holds together, and should be somewhat more substantial in a bridge carrying 138-ton engines than in one carrying 90-ton engines. The theorist cannot estimate the amount of material necessary for these primary conditions, nor can the manufacturer realize what they are, as he seldom sees his output in actual service. The least amount of bracing to be used in any case is, therefore, a matter of judgment based upon field observation, and every specification which aims at good results should fix definitely the

Mr. Greiner. least amount of bracing required, no matter whether the wind blows with a pressure of 30 lbs. per square foot or not.

Some provision should be made for elevating the outer rail on curves, as this can be done in a variety of ways, some of which are good, but expensive, while others are vicious, but cheap. The contractor should not have the privilege of inserting a rectangular raising piece under the outer rail, which he will probably do unless the engineer specifies otherwise.

A specification should be consistent. If some of the best types of bridge trusses are to be barred out because they have members subjected to alternate stresses, there is no necessity for describing in detail the manner in which the sections of such members are to be determined. If a unit stress of 1 400 lbs. is permitted in cross-ties resting upon stringers spaced 7 ft. apart, why should this stress be reduced to 1 000 lbs. when the stringers are a few inches farther apart? As the writer understands, operating machinery, including gearing and wedges, is a part of a certain type of bridge, and is usually made of cast steel. Why, then, should it be specified that the structure shall be wholly of rolled steel and wrought iron; and, if such a structure must be made wholly of rolled steel and wrought iron, why specify the quality of cast iron? If the specifications are for "steel railroad bridges" as given in the title, what is the use of describing the character of the wrought iron to be used, especially when no provisions are made for unit stresses?

There should be no ambiguity or uncertainty concerning column formulas. The author gives the formula

$$P = p \left(1 + \frac{l^2}{18\,000\,r^2} \right)$$

for finding the increased stress in a column, due to flexure, in cases where the member is subjected to alternate tension and compression, and states that this increased stress is to be added to the tensile stress, the resulting unit stress not to exceed that specified for live load. It is not positively clear whether he intends to use a tensile or compressive unit stress, as both are specified for live stresses. If the ordinary column formula

$$P = \frac{18\,000}{1 + \frac{l^2}{18\,000\,r^2}}$$

is sufficient to provide for compression alone, and the ordinary unit stress for tension alone, why should it be necessary to increase the compression stress when the column is also subjected to tension? The maximum tension makes the maximum compression neither more nor less, and the writer would prefer to proportion the area for each kind of stress separately and then add the two areas thus found for the required area of the sections.

There are provisions in these specifications quite at variance with Mr. Greiner. the writer's practice, but as they are mainly questions of opinion, discussion would be useless. Personally, the writer has more confidence in physical tests than in chemical analyses, and believes that a better knowledge of the quality of the steel can be obtained by punching, drifting, cold, hot and quench-bending tests than by the most careful chemical analysis. The analyses indicate physical properties not always obtained, while severe physical tests demonstrate practically whether or not the metal is good, indifferent or bad. The writer's practice is to limit the phosphorus only and to rely on the physical tests for defects.

EDGAR MARBURG, M. Am. Soc. C. E.—This paper opens the way Mr. Marburg. once more to a discussion of the conclusions to be drawn from Wöhler's famous experiments and later ones of a like general character. The subject has been presented, however, from a somewhat novel point of view, in that the author attempts to account, on rational grounds, for certain coincidences which, so far as they have been observed at all, seem to have been regarded heretofore as purely accidental.

In order to first determine independently how nearly the approximate constancy of what the author terms the "equivalent dead load" [Min. + 2 (Max. — Min)] might be borne out by Wöhler's results, the writer was led to undertake a careful examination of all accessible literature on the subject. In this examination a number of important errors and disagreements were discovered, which the writer thinks may well be brought to more general notice. They are, in fact, for the most part, pertinent to this discussion.

Wöhler's summary of stress limits, within which rupture was not produced after a vast number of repetitions, as quoted by Bauschinger,* is as follows, in terms of centners per German square inch:

	Max. stress.	Min. stress.
Wrought Iron.—I. Phoenix axle iron, supplied in 1857.	+160	—160
	+300	0
	+440	+240
Cast Steel.— II. Untempered spring steel.....	+500	0
	+700	+250
	+800	+400
	+900	+600
III. Krupp axle steel supplied in 1862	+280	—280
	+480	0
	+800	+350

With the exception of the test on wrought iron, between the limits of +440 and +240, these values were obtained from bending tests, the modulus of rupture being computed by the common formula for flexure.

* "Bauschinger's Communications" (1886), Vol. xiii, p. 44.

Mr. Marburg. The above quantities are given also by Spangenberg,* with the addition of the following shearing values, computed from torsion tests on cast steel, by the common formula for torsion:

	Max. stress.	Min. stress.
IV. Krupp axle steel (cast).....	+220	-220
	+380	0

In these last two tests, failure did not ensue after 23 850 000 and 19 100 000 repetitions, respectively.

The foregoing values, reduced to pounds per English square inch, are collected in Table No. 3. In the last column the values given in *Engineering*† for corresponding tests, as quoted in part by the author, are shown.

TABLE No. 3.

Material.	Group.	STRESSES.		VALUES FROM <i>Engineering</i> .	
		Maximum.	Minimum.	Maximum.	Minimum.
Wrought iron.....	I	+16 600 +31 200 +45 700	-16 600 0 -25 000	+17 120 -35 310 +47 080	-17 120 0 -25 080
Spring steel.....	II	+52 000 +72 800 +83 200 +93 600	0 +26 000 +41 600 +62 400	+53 500 +74 900 +85 600 +96 300	0 +26 750 +42 800 +64 200
Axle steel.....	III	+29 100 +49 900 +83 200	-29 100 0 +36 400	+29 960 -51 360 +85 600	-29 960 0 +37 450
Axle steel.....	IV	+23 900 +39 500	-23 900 0	+23 540 +40 660	-23 540 0

The value 35 310 for wrought iron, as given in *Engineering*, is apparently a typographical error, as will be shown presently. All other values from that journal seem to be about 3% in error.‡ This observation applies also to Unwin's tables.§

* "The Fatigue of Metals," pp. 13 and 14.

† Vol. xi (1871), p. 397.

‡ In *Engineering* (p. 200) the value of the German centner is placed at 113 436 lbs. English, for which the writer can find no authority. Its true value seems to be 110.23 lbs. English (50 kg.), as given in the Table of Weights of the "Standard Dictionary," and in the English translation of Spangenberg's "Fatigue of Metals" (p. 12). The metric pound (500 grams) has been the commercial standard of weight throughout Germany only since 1872, whereas Wöhler's experiments were made at Frankfort between the years 1859 and 1870. However, the metric pound has been the legal standard for customs since 1840, was adopted by the railroads in 1851, and its use became more and more general until 1872, when it became the universal standard throughout the empire. The old Prussian pound was equivalent to 467.7 grams (Johnson's and Brockhaus' Encyclopædias). While 110.23 lbs. appears to be the probable value of the centner quoted in these tests, the question here is one of relative rather than absolute values.

The German inch is equivalent to 1.0297 English inches.

§ "The Testing of Materials of Construction." Unwin's values (in gross tons per English square inch) seem to have been taken directly from *Engineering*. The wrong value (35 310) for wrought iron appears also in Unwin.

The value 35 310, which has been designated a typographical error, Mr. Marburg is the quantity concerning whose origin the author expresses some doubt. That it is a misprint is attested, not only by the agreement of the corresponding values (31 200) quoted by Bauschinger and Spangenberg, but this value is given also by Weyrauch.* Moreover, the error is indicated by the context in *Engineering*, in which Wöhler's reason for adopting 31 200 (given as 32 100) is set forth. For precisely the same reason the value 34 240, assumed instead by the author, appears inadmissible, namely, that the tension tests resulted in rupture under this load. Wöhler's value was derived from the bending tests on the same material.

The quantities in Table No. 3 are all correctly given by Professor S. W. Robinson, M. Am. Soc. C. E.,† with the exception of the third value for wrought iron, which he placed at 41 600 (400 centners) instead of 45 700 (440 centners). This error is apparently attributable to a misprint in the translation from Spangenberg.‡

Concerning the strength of the three materials named, under static loads, there appears to be considerable uncertainty. Bauschinger, quoting directly from Wöhler, places the average tensile value at 445 centners (range 440 to 450), or 46 300 lbs. It is not clear how this can be reconciled with the higher initial stress of 49 900 lbs. per square inch (480 centners), repeated 800 times before rupture. Presumably the great rapidity with which the stresses were alternately developed and released did not allow a sufficient time interval for rupture at a single application, even for loads somewhat in excess of the static strength. The static tensile strength of the steel axles, Bauschinger places at 1 040 centners, or 108 100 lbs., and states that he is unable to find this factor for the spring steel in Wöhler's publications.

Weyrauch quotes for static tests on Phoenix iron, 57 200 lbs. (4 020 kg. per square centimeter) in bending, and 46 800 lbs. (3 290 kg. per square centimeter) in tension. The value, 46 800 lbs., agrees with the upper limit (450 centners) given by Bauschinger. The considerably higher value found in bending may be attributed to the inaccuracy of the common formula of flexure for stresses exceeding the elastic limit. The static strength of the axle steel, Weyrauch places at 104 400 lbs.‡ (7 340 kg. per square centimeter).

Concerning the static strength of the spring steel, Weyrauch writes: "Wöhler found for this steel in (static) bending tests $t = 1\ 100$ centners (114 400 lbs.)."

In *Engineering* the following are given as the experimental values in tension under static loads:

* Given as 2 195 kg. per square centimeter, "Iron and Steel Constructions," pp. 43 and 44.

† *Transactions*, Am. Soc. C. E., June, 1886, Vol. xv., p. 439.

‡ The correct value, 440 centners, appears on pp. 13 and 43, but, though a misprint, it is given as 400 centners on p. 41.

§ Whether for tension or bending does not appear. The former seems probable.

Mr. Marburg. Phoenix axle iron (supplied in 1857) .. $\left\{ \begin{array}{l} 47\ 080 \\ 48\ 150 \end{array} \right\}$ average, 47 615 lbs.

Krupp axle steel (supplied in 1862) .. $\left\{ \begin{array}{l} 109\ 675 \\ 112\ 350 \\ 112\ 350 \end{array} \right\}$ average, 111 460 lbs.

These averages, corrected for the apparent error of about 3%, previously mentioned, become 46 200 and 108 200; or essentially the same as given by Bauschinger.

From the above data, Table No. 4 may be constructed.

TABLE No. 4.—STATIC STRENGTH FROM WÜHLER'S TESTS, IN POUNDS PER SQUARE INCH.

Authors.	GROUP I.—AXLE IRON.		GROUP II.—SPRING STEEL.		GROUPS III AND IV.—AXLE STEEL.	
	Tension.	Bending.	Tension.	Bending.	Tension.	Bending.
Bauschinger.....	46 300				108 100	
Weyrauch.....	46 800	57 200		114 400	104 400	
Engineering.....	46 200				108 200	
Probable values..	46 300	57 200		114 400	108 100	

The values in Table No. 3 and the probable values in Table No. 4 may be combined in Table No. 5, according to the scheme adopted by the author:

TABLE No. 5.

Material.	Group.	WÜHLER'S TESTS.		Live stress. (Max.—Min.)	Dead stress.	Equivalent dead stress. (Dead + 2 live.)
		Maximum.	Minimum.			
Wrought iron. }	I	+ 16 600	— 16 600	33 200	0	* 66 400 (bending).
		+ 31 200	0	31 200	0	* 62 400 "
		+ 45 700	+ 25 000	20 700	25 000	* 66 400 (tension).
		+ 46 300	+ 46 300	0	46 300	46 300 "
		+ 57 200	+ 57 200	0	57 200	57 200 (bending).
Spring steel. }	II	+ 52 000	0	52 000	0	* 104 000 (bending).
		+ 72 800	+ 26 000	46 800	26 000	* 119 600 "
		+ 83 200	+ 41 600	41 600	41 600	* 124 800 "
		+ 98 600	+ 62 400	31 200	62 400	* 124 800 "
		(+ 104 000)	(+ 68 600)	(35 400)	(68 600)	(139 400) "
		+ 114 400	+ 114 400	0	114 400	114 400 "
Axle steel. }	III	+ 29 100	— 29 100	58 200	0	116 400 (bending).
		+ 49 900	0	49 900	0	99 800 "
		+ 83 200	+ 36 400	46 800	36 400	130 000 "
		+ 108 100	+ 108 100	0	108 100	108 100 (tension).
Axle steel. }	IV	+ 22 900	— 22 900	45 800	0	91 600 (shearing).
		+ 39 500	0	39 500	0	79 000 "
		+ 108 100	+ 108 100	0	108 100	{ 85 400 " 108 100 (tension).

The fifth set of results for Group II, enclosed in parentheses, corresponds to the author's set No. 2. The reason for its exclusion by Wöhler is not apparent, except that the result seems anomalous, compared with the other tests on this material, and that the number of repetitions (19 673 000) is about half as great as for the remaining four sets. The rejection of the third set of values for wrought iron, between the limits of + 45 700 and + 25 000 would appear better warranted, for in this case there were only 4 000 000 repetitions.

The static strength of the axle steel (108 100) appears only for tension, and is probably lower than for bending. The static strength in shearing for Group IV (85 400) was estimated by multiplying the above value found in tension by 0.79, this being approximately the ratio between the shearing and bending values, as may be seen from a comparison of the first two sets in Groups III and IV. This value is therefore also probably too low.

The values in the last column, distinguished by asterisks, are those given by the author, after correction.

From a review of the quantities in the last column, it appears that the experiments give no indication of a law by which the sum, Min. + 2 (Max. — Min.) is even approximately constant for each of the several groups. Although, as the author observes, Wöhler's results for Groups I and II "are more complete and satisfactory than the others," they deserve to be so

regarded (especially Group II), by reason of their range, and because the metal in Group I was forged wrought iron. There is no apparent reason, however, for considering the individual results in Groups III and IV as in any way less trustworthy, much less for disregarding them entirely in an inquiry of the kind instituted by the author.

Again, it is to be remembered that Wöhler's results are at best rather loose approximations. The difference between the load by which rupture was not produced after many million applications and the next higher one which caused failure was frequently considerable. The true value is an intermediate one whose magnitude can only be conjectured. The reasoning applied by the author to sets Nos. 1 and 5 is no less applicable to set No. 3.

The whole situation is clearly shown on the diagram, Fig. 5, drawn after the method of Weyrauch. The ordinates from the 0.0 axis to the straight line ab represent minimum stresses. The correspond-

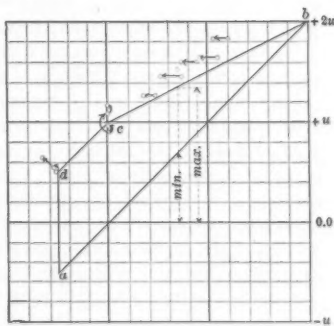


FIG. 5.

Mr. Marburg. ing maxima are laid off along the same ordinates. The experimental values in Table No. 5 are plotted by placing the static strength for each of the several groups equal to $2u$. For Group I the static value in bending was used as a basis.* In some cases several values are coincident or sensibly so, of which the diagram gives no indication. The arrows connecting two plotted values indicate the shifted position of the point in question by considering the next higher load which caused failure. The true value lies somewhere between points thus connected. For Group III the data were not available.

To exactly fulfill the law suggested by the author, all points should lie on the line $b c$, for stresses of like character, and on $c d$ for reversed stresses. The actual departure from such a law is, however, not sufficiently emphasized in this diagram, for, in the formula $c = \text{Min.} + 2 (\text{Max.} - \text{Min.})$, the true difference between the value of c for any given experiment and the hypothetical constant value $2u$ is twice as great as the vertical intercept between $b c$ or $c d$ and the point which marks the experimental value considered.

However Wöhler's results may be viewed, they admit of no inferences concerning the "theory of work." In the absence of records of deformations, the "work" remains an unknown quantity. It is true that, within the elastic limit, a load suddenly applied, without impact, develops theoretically twice the unit stress produced by the same load gradually applied; but even within the elastic limit this familiar law can find no application here. It is to be remembered that Wöhler's tests were not made by the application and release of known loads, in which case the stresses would have varied with the rapidity of the operations, and would have been different above and below the elastic limit, with other conditions constant. The apparatus was, on the contrary, so adjusted by means of calibrated springs that the stresses themselves remained sensibly constant. This distinction is an exceedingly important one.

Again, there appears to be no ground, in Wöhler's experiments, or any others known to the writer, which would tend to cast doubt, or, as the author asserts, "entirely destroy the theory of the perfect elasticity of metals," it being understood that the accepted theory within the primitive elastic limit is here referred to. This theory found its most triumphant vindication at the hands of Bauschinger himself, whose conclusions the author appears to have somewhat misconstrued. By means of his remarkable measuring apparatus—sensitive to the ten-thousandth part of a millimeter (about $\frac{2}{250,000}$ in.)—Bauschinger not only confirmed the correctness of Hooke's classic law, but he showed that within the elastic limit the permanent set, as well as the

* Since this matter has gone to press, the writer's attention has been called to the fact that the static value in tension would have been a more suitable basis, inasmuch as the fiber stresses in bending are within the elastic limit. This would lead to a more marked divergence for this group, from the law suggested by the author, than is shown in the diagram.

total elongation remained constant, both after the lapse of time and Mr. Marburg. under repeated applications of the original load.*

Accordingly, after an initial stress, of a given magnitude within the elastic limit, has been once developed, the material is afterward perfectly elastic up to the limit of that stress; that is to say, complete and instant recovery of form upon release of stress is then assured.

It was, indeed, on these observations that Bauschinger based his claim that stresses within the elastic limit, no matter how frequently repeated, could not be expected to cause rupture. He showed, moreover, that, through the elevation of the elastic limit, this was true also, with certain limitations, for stresses in excess of the primitive elastic limit.† In case, however, the intensity of stress exceeded a certain higher limit, successive repetitions increased the deformation, so that ultimate rupture became then inevitable. The seeming contravention of these laws in the case of reversed stresses has been met by Bauschinger's ingenious theory, recognizing the distinction between the primitive and what he terms the "natural" elastic limit, thus showing that the apparent anomaly is but a paradox.

The author affirms his faith in the "theory of the fatigue of metals," without defining the sense in which this observation is intended. While the signification of "fatigue" is not infrequently misconstrued, and the selection of this particular term may, in the light of present knowledge, have been ill-advised, there appears to be now no reasonable ground for differences of opinion concerning the phenomena for which it stands. The matter has, indeed, long since become one of fact rather than theory. Bauschinger's investigations have apparently furnished conclusive proof that the metal in specimens subjected to oft-repeated tensile stresses experiences no sensible change, either in its molecular structure or in its subsequent behavior under static loads.‡ A theory of fatigue involving an assumed alteration in the physical characteristics of the metal appears then as chimerical as the time-worn theory of cold crystallization. The writer does not wish to be understood as inferring that the author holds any such views, but that they are yet all too prevalent among engineers will probably be admitted. On the other hand, a denial of the reality of such a thing as "fatigue"—understood simply as meaning that rupture will be produced under certain conditions by stresses far within the static strength of a given material—no longer deserves to be entertained seriously.

Returning now to Wöhler's tests, the chief interest attached to them, in the opinion of the writer, is to be found in the fact that they

* "Bauschinger's Communications" (1886), Vol. xiii, pp. 14 and 15.

† *Ibid.*, pp. 16 and 39.

‡ In general, the static tensile strength was somewhat raised by oft-repeated stresses. The difference, in every case, was relatively small. The endurance tests in bending, made at the Watertown Arsenal, exhibited, however, in some cases considerable irregularity in his respect.

Mr. Marburg. proved (a) that innumerable repeated stresses of like character, not exceeding the primitive elastic limit, cannot produce rupture, and (b) that the safe limit for equal stresses, opposite in character, is far below the primitive elastic limit. The former discovery served as a welcome reassurance to the users of metal, the latter as a wholesome warning, despite the fact that the trying condition of reversed stresses had, it seems, been recognized intuitively by American engineers before Wöhler's announcement, and provided for by various methods.

The writer is not in sympathy, however, with the efforts that have been made toward the introduction of so-called "fatigue formulas" into specifications for structural work. This practice is believed to be founded on wrong principles. The added labor it involves—in this the writer speaks from a somewhat extended experience in bridge practice—is to be justified only on grounds of superior scientific excellence, which claim, he thinks, cannot be satisfactorily demonstrated. The writer must refrain, however, from elaborating his views on this point, the contemplated limits of this discussion having already been far exceeded.

Mr. Wilson. JOSEPH M. WILSON, M. Am. Soc. C. E.—It is now more than thirty years since the question of specially providing for the effect of live load on a bridge structure, on different conditions from those of dead load, began to receive serious consideration in America, and much longer than that (nearly fifty years), since such effect was first noticed.

Rankine* states that:

"The additional strain arising, whether from the sudden application or swift motion of the load, is sufficiently provided for in practice by the method already so frequently referred to, of making the factor of safety for the traveling part of the load about double the factor of safety for the fixed part."

It was the custom, even then, to use a higher allowable stress in roofs, and such structures, than in bridges.

Fairbairn,† gives quite a series of tables and experiments to determine the effect of impact, vibratory action and long-continued changes of load on wrought-iron girders.

Unwin‡ states that:

"In the experiments of the Royal Commission on Railway Structures in 1849, when a truck weighing 1 120 lbs. was run over a pair of light cast-iron bars at a velocity of 30 miles an hour, the maximum deflection was twice as great as when the same load quietly rested on the bars." "With 2 066 lbs. the dynamical deflection was three times as great as the statical deflection. But the theory deduced from the experiments led Professor Willis to the conclusion that effects of this kind, produced by a rolling load, and alarmingly manifested in experi-

* "Manual of Civil Engineering," Third Edition, 1864, p. 278 (perhaps also in earlier editions).

† "Useful Information for Civil Engineers," Third Series, London, 1866. *Philosophical Transactions*, 1864, p. 311.

‡ "Wrought-Iron Bridges and Roofs," London, 1869.

ments on a small scale, would be so greatly diminished with bridges Mr. Wilson. of large dimensions as to be of comparatively little importance."

It was, therefore, recognized at that early date that short-span bridges demanded more attention in this respect than those of long span. Unwin also states that:

"In experiments on bars loaded and unloaded alternately, they found that if the deflection did not exceed that due to a statical load amounting to one-third of the breaking weight, the bars suffered 10 000 repetitions of load without appreciable injury; but one bar broke after 50 000 repetitions of load; and when the deflection was increased so as to be equivalent to that produced by a dead load of one-half the breaking weight, all the bars broke after a greater or less length of time."

From these various experiments combined with theory and experience, Unwin deduces* that "for practical purposes, a rolling load may be assumed to be equivalent to a dead load of twice its magnitude," and states that:

"(1) The load may be reduced to an equivalent dead load, whose magnitude will be twice the actual live load plus the dead load," * * * taking "a limiting stress suitable to a dead load." Or:

"(2) The stresses may be calculated from the actual load, considered as a dead load, and a variable limiting stress may be adopted, dependent on the ratio of the dead to the live load."

Unwin submits a table, giving such values for the limiting stress as would be equivalent to allowing twice as much metal for a given live load, as for an equal dead load, as follows:

SAFE LIMITS OF STRESS.

Ratio of live load to dead load.	TONS PER SQUARE INCH.	
	Tension.	Compression.
All dead load.....	7.00	5.50
.25.....	5.83	4.59
.33.....	5.60	4.40
.50.....	5.25	4.12
.66.....	5.00	3.93
1.00.....	4.66	3.66
2.00.....	4.20	3.30
All live load.....	3.50	2.75

As to the practice at that time, Unwin states:

"Although the method of proportioning bridges just indicated is really the scientific method, it has not hitherto been often followed in practice. The plan adopted has been to calculate the stresses due to the total load as if quiescent, and to proportion all bridges to a stress lying between the limits which have just been assigned for dead and live loads respectively."

The writer worked on this basis for short-span bridges as early as 1870-71, and it appears to have been adopted by many bridge engineers, and has continued in use by some up to the present day.

* Pp. 36-37, same edition.

Mr. Wilson. Wöhler's published experiments and Launhardt's formula came later, and were accepted by the writer as furnishing a more rational and proper method of solving the question than anything that had as yet been produced. Wöhler's experiments followed the same line, and confirmed the indications of those previously made, adding materially to the experience which, after all, is what must be depended upon in such matters. The writer is not aware of any later experiments or results to disprove them or to suggest different conclusions. The best that can be said of other methods of procedure is that they conform practically, or nearly so, to that of Launhardt.

The first printed specification by the writer, in which the use of Launhardt's formula appeared, was in 1882, and although he had adopted and used it previously, at least as early as 1880, there was no occasion for its earlier publication, as all calculations and detail drawings for bridges were at that time prepared by him in complete shape ready for bidders, and the method of calculation did not enter into the specification.

The author's specification is commendable in that it admits the value of Wöhler's experiments and Launhardt's formula, and is an effort to simplify calculations without departing materially from the same results. His conclusions are certainly very interesting, but, if he will pardon the writer for drawing attention to the matter, the results are precisely the same as produced by the method proposed by Unwin in 1869 [see (1)].

The writer is not prepared, as yet, to abandon the method he has used since 1880, in the adoption of the Launhardt formula, although should anything be suggested, showing substantial evidence of being really better, he would be very glad to give it favorable consideration. What is needed is a thorough and systematic series of experiments, on a liberal scale, conducted by competent engineers with ample facilities and funds, to establish by actual experience a reliable theory of the action of live load and impact. He joins heartily with Mr. Waddell and Mr. Bouscaren* in this matter.

No amount of pure theoretical discussion will determine it; and as for the experience that engineers talk of, how many of them ever see their bridges after they are up? They design them in the shop and that is the end of their experience. The writer is not referring to the railroad engineer, who does have opportunity to examine his bridges and note their action under service, if he is competent in this line of work; but to the shop engineer who seldom gets the benefit of this experience. The writer thoroughly agrees with the author on the subject of bridge engineers connected with the railroad system, and a periodical inspection of bridges by a thoroughly competent, theoretical and practical engineer.

* *Transactions, Am. Soc. C. E.*, Vol. xxvi, pp. 182 and 276.

It may be a curious thing to say, but each bridge, like many other Mr. Wilson. objects in animate and inanimate nature, has its individual peculiarities, and requires to be studied in its action, which can only be accomplished by periodical inspections. The engineer of bridges should be entirely competent to do this, to prepare specifications, make calculations and inspect material, even if he does not actually make the designs, and to act as an expert for the railroad company in all matters connected with his department, including the criticism and reporting on designs and details of bridges that may be submitted in competition by bridge companies.

The writer has never been able to understand why engineers have so much fear of the labor involved in the use of Launhardt's formula. A table of working stresses, such as Table No. 6, would go far toward relieving such apprehension. From this table the required sections are calculated. The details are then designed directly from the sections of the connected members; for instance, the bearing on pins would equal one and one-half times the total section of the member, etc.

For compression members the permissible stress thus found is reduced to allowable working stress by multiplying by the percentage given in Table No. 7 for struts. The form of this table is peculiarly adapted to labor saving. The value of $\frac{l}{d}$ is always quickly found;

the value of $\frac{d^2}{r^2}$ is found to be approximately constant for each different shape of cross-section, as shown in the notes attached to the table, although where accurate results are required this value is usually calculated by working out the moment of inertia of the actual section of the member (particularly for upper chords and vertical struts), the value of r^2 being deduced in terms of d^2 .

The application of the proper formula by the use of the table then becomes a very simple matter.

The writer heartily agrees with those who would reduce and simplify the labor of calculation, provided equally reliable results are obtained—results that can be depended upon—but he has seen engineers, otherwise trustworthy and competent, fall into grievous trouble by the use of shorthand methods of computation, where a little care would have saved, not only reputation, but also an immense amount of annoyance to superior officers.

The writer notes the author's remarks on the subject of elevated railways and large railway terminal approaches, where the passage of trains is almost continuous, and agrees that such conditions require special attention. Experience is a valuable teacher, and experience in such cases only dates back in America for comparatively a few years. Suburban travel has increased enormously during that time, and bridges designed on first-class specifications, thoroughly able to carry main-line traffic of the heaviest kind, certainly do appear to

Mr. Wilson. suffer fatigue and wear out under the frequent repetitions of the same load which they receive when placed in such locations as mentioned.

The writer desires to mention here for the benefit of the profession an experience with pin-connected triangular trusses, having alternate compressive and tensile stresses in the same members, and acting under continuous service as here spoken of, which would lead to the conclusion that this type should not be used, at least in such locations. The rapidly occurring alternate stresses on the braces appeared to cause the pins to revolve in place so that they were worn out very rapidly in deep grooves under the bearings. The pins were calculated on the most liberal allowance for shear and bending moments, according to modern requirements, and the result would have been hard to believe had it not been seen. The writer has seen pins in old bridges of the Linville type, of diameters very considerably less than the present specifications would allow, sustaining severe bending stress and certainly much overstrained (this strain being only in one direction), and standing up to their work perfectly well without showing in the least such wear as above noted.

The writer agrees with the author that it is the live strain which causes loose rivets, and which wears out the bridge.

All riveted connections should have plenty of rivets, and they should be tight. This shows itself forcibly on terminal bridges.

FORMULAS FOR WORKING STRESSES (SEE TABLE No. 6):

Acting in Single Direction.

$$a = u \left(1 + \frac{t - u}{u} \times \frac{\text{Min. } B}{\text{Max. } B} \right)$$

$$\text{when } t = 2u, \quad a = u \left(1 + \frac{\text{Min. } B}{\text{Max. } B} \right).$$

$$c = \frac{a}{1 + \frac{l^2}{5000 w^2}}, \quad \left(\text{when } \frac{l}{w} = 12 \right).$$

t = stress for all dead load.

u = stress for all live load.

Acting in Opposite Directions Alternately.

$$a = u \left(1 - \frac{u - s}{u} \times \frac{\text{Max. } B'}{\text{Max. } B} \right)$$

$$\text{when } s = \frac{1}{2}u, \quad a = u \left(1 - \frac{1}{2} \times \frac{\text{Max. } B'}{\text{Max. } B} \right).$$

s = stress for equal tension and compression.

u = stress for either tension or compression alone.

The greater stress upon member = Max. B , and the lesser stress (in opposite direction) = Max. B' .











TABLE No. 6.—WORKING STRESSES, IN NET TONS PER SQUARE INCH. Mr. Wilson.

ACTING IN SINGLE DIRECTION.*					ACTING IN OPPOSITE DIRECTIONS,* ALTERNATELY.				
Min. B. Max. B.	Tension.		Compression.		Tension.		Compression.		Max. B' Max. B.
	Value of "a."		Value of "a."	Value of "c."	Value of "a."		Value of "a."		
	For high-test tension iron for Max. B.	For ordinary tension iron for Max. B.	For compression, one diameter for Max. B.	For compression (12 diameters) upper flanges pl. girder, braced.	For high-test tension iron for Max. B.	For ordinary tension iron for Max. B.	For compression, one diameter for Max. B.		
All live									
.00	3.750	3.50	3.250	3.162	3.750	3.500	3.250	.00	
.02	3.825	3.57	3.315	3.225	3.713	3.465	3.218	.02	
.04	3.900	3.64	3.380	3.289	3.675	3.430	3.185	.04	
.06	3.975	3.71	3.445	3.352	3.638	3.395	3.153	.06	
.08	4.050	3.78	3.510	3.415	3.600	3.360	3.120	.08	
.10	4.125	3.85	3.575	3.478	3.563	3.325	3.088	.10	
.12	4.200	3.92	3.640	3.542	3.525	3.290	3.055	.12	
.14	4.275	3.99	3.705	3.605	3.488	3.255	3.023	.14	
.16	4.350	4.06	3.770	3.668	3.450	3.220	2.990	.16	
.18	4.425	4.13	3.835	3.731	3.413	3.185	2.958	.18	
.20	4.500	4.20	3.900	3.795	3.375	3.150	2.925	.20	
.22	4.575	4.27	3.965	3.858	3.338	3.115	2.893	.22	
.24	4.650	4.34	4.030	3.921	3.300	3.080	2.860	.24	
.26	4.725	4.41	4.095	3.984	3.263	3.045	2.828	.26	
.28	4.800	4.48	4.160	4.048	3.225	3.010	2.795	.28	
.30	4.875	4.55	4.225	4.111	3.188	2.975	2.763	.30	
.32	4.950	4.62	4.290	4.174	3.150	2.940	2.730	.32	
.34	5.025	4.69	4.355	4.237	3.113	2.905	2.698	.34	
.36	5.100	4.76	4.420	4.301	3.075	2.870	2.665	.36	
.38	5.175	4.83	4.485	4.364	3.038	2.835	2.633	.38	
.40	5.250	4.90	4.550	4.427	3.000	2.800	2.600	.40	
.42	5.325	4.97	4.615	4.490	2.963	2.765	2.568	.42	
.44	5.400	5.04	4.680	4.554	2.925	2.730	2.535	.44	
.46	5.475	5.11	4.745	4.617	2.888	2.695	2.503	.46	
.48	5.550	5.18	4.810	4.680	2.850	2.660	2.470	.48	
.50	5.625	5.25	4.875	4.743	2.813	2.625	2.438	.50	
.52	5.700	5.32	4.940	4.807	2.775	2.590	2.405	.52	
.54	5.775	5.39	5.005	4.870	2.738	2.555	2.373	.54	
.56	5.850	5.46	5.070	4.933	2.700	2.520	2.340	.56	
.58	5.925	5.53	5.135	4.996	2.663	2.485	2.308	.58	
.60	6.000	5.60	5.200	5.060	2.625	2.450	2.275	.60	
.62	6.075	5.67	5.265	5.123	2.588	2.415	2.243	.62	
.64	6.150	5.74	5.330	5.186	2.550	2.380	2.210	.64	
.66	6.225	5.81	5.395	5.249	2.513	2.345	2.178	.66	
.68	6.300	5.88	5.460	5.313	2.475	2.310	2.145	.68	
.70	6.375	5.95	5.525	5.376	2.438	2.275	2.113	.70	
.72	6.450	6.02	5.590	5.439	2.400	2.240	2.080	.72	
.74	6.525	6.09	5.655	5.502	2.363	2.205	2.048	.74	
.76	6.600	6.16	5.720	5.566	2.325	2.170	2.015	.76	
.78	6.675	6.23	5.785	5.629	2.288	2.135	1.983	.78	
.80	6.750	6.30	5.850	5.692	2.250	2.100	1.950	.80	
.82	6.825	6.37	5.915	5.755	2.213	2.065	1.918	.82	
.84	6.900	6.44	5.980	5.819	2.175	2.030	1.885	.84	
.86	6.975	6.51	6.045	5.882	2.138	1.995	1.853	.86	
.88	7.050	6.58	6.110	5.945	2.100	1.960	1.820	.88	
.90	7.125	6.65	6.175	6.008	2.063	1.925	1.788	.90	
.92	7.200	6.72	6.240	6.072	2.025	1.890	1.755	.92	
.94	7.275	6.79	6.305	6.135	1.988	1.855	1.723	.94	
.96	7.350	6.86	6.370	6.198	1.950	1.820	1.690	.96	
.98	7.425	6.93	6.435	6.261	1.913	1.785	1.658	.98	
All dead 1.00	7.500	7.00	6.500	6.324	1.875	1.750	1.625	1.00	

* For formulas see page 232.

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TABLE NO. 7—(Continued).

Average Values of $\frac{d^3}{l^3}$	For Light Weight	For Heavy Weight	Average Values of $\frac{r^3}{l^3}$	For Light Weight	For Heavy Weight
Even Legged Angles.	26.0	26.4		16.0	
Uneven " "	10.0	10.0		9.0	
" " "	11.7	11.7		12.0	
Channel—2" to 12"	6.8	6.8		6.8	
" 13"	7.4			16.13	
" 2" to 16"	12.7	12.7		8.9	
Deck Beams—4" to 10"	7.1	7.1		10.4	
" " "	35.0	35.0			
" " "	6.4	6.4		6.5	8.00
" " "	21.6	22.5		7.0	8.00

Select the proper value of $\frac{l}{d}$ in the left-hand column of the table and follow a horizontal line from this until it reaches a vertical column, at the top of which is the proper value of r^3 in terms of d^3 , and there will be found at the intersection a percentage-value with which to multiply a , to obtain b .

$$*b = \frac{a}{1 + \frac{1}{36,000} r^2}$$

$$+b = \frac{a}{1 + \frac{1}{24,000} r^2}$$

$$\ddagger b = \frac{a}{1 + \frac{1}{18,000} r^2}$$

a = permissible stress previously found,
 b = allowable working stress per square inch.

l = length of piece in inches, center to center of connections,
 r = least radius of gyration of the section, in inches.

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TABLE No. 7.—STRENGTH OF WROUGHT-IRON STRUTS, PENNSYLVANIA RAILROAD SPECIFICATION.

BOTH ENDS FIXED*																	
Square of least radius of gyration of cross-section (r^2) =																	
l	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2
d	6	7	8	9	10	12	14	16	18	20	24	28	32	36	40	44	48
12	977	973	969	965	962	954	947	940	933	926	913	899	887	874	862	850	837
13	1073	1069	1064	1060	1056	1046	1037	1028	1020	1012	998	984	970	956	942	928	914
14	1173	1168	1163	1158	1153	1141	1129	1117	1104	1091	1075	1060	1045	1030	1014	998	982
15	1278	1272	1266	1261	1255	1241	1227	1213	1198	1183	1165	1148	1131	1114	1096	1078	1060
16	1387	1380	1373	1366	1359	1343	1327	1311	1294	1276	1257	1238	1219	1200	1181	1161	1141
17	1500	1492	1484	1476	1467	1449	1431	1412	1392	1371	1350	1329	1308	1286	1264	1242	1220
18	1617	1608	1600	1591	1581	1562	1542	1521	1500	1477	1454	1431	1408	1384	1360	1336	1312
19	1738	1728	1719	1709	1698	1678	1656	1633	1610	1585	1560	1535	1510	1484	1458	1431	1404
20	1863	1852	1842	1831	1820	1799	1775	1751	1726	1700	1673	1645	1617	1589	1560	1531	1502
21	1992	1980	1969	1957	1945	1923	1898	1872	1845	1817	1788	1758	1728	1697	1666	1634	1602
22	2125	2112	2100	2087	2074	2051	2025	1998	1970	1941	1910	1878	1846	1813	1779	1745	1711
23	2262	2248	2234	2220	2206	2182	2155	2127	2098	2068	2035	2001	1967	1932	1897	1861	1825
24	2403	2388	2373	2358	2343	2318	2290	2261	2231	2199	2165	2130	2094	2058	2021	1984	1946
25	2548	2532	2516	2500	2484	2458	2429	2399	2368	2335	2299	2262	2225	2187	2149	2110	2071
26	2697	2680	2663	2646	2629	2599	2569	2538	2506	2473	2435	2397	2358	2319	2279	2238	2197
27	2850	2832	2814	2796	2778	2747	2716	2684	2651	2617	2578	2538	2498	2457	2415	2373	2330
28	3007	2988	2969	2950	2931	2899	2867	2834	2799	2764	2724	2683	2641	2599	2556	2513	2469
29	3168	3148	3128	3108	3087	3055	3022	2988	2953	2917	2876	2834	2791	2748	2704	2660	2615
30	3333	3312	3291	3270	3249	3216	3182	3147	3111	3074	3032	2989	2945	2900	2855	2810	2764
31	3502	3480	3458	3436	3413	3379	3344	3308	3271	3233	3190	3146	3101	3055	3009	2962	2915
32	3675	3652	3629	3606	3582	3548	3512	3475	3437	3398	3354	3309	3263	3216	3169	3121	3073
33	3852	3828	3804	3780	3755	3720	3684	3647	3609	3569	3524	3478	3431	3383	3335	3286	3237
34	4033	4008	3983	3958	3932	3896	3859	3821	3783	3744	3698	3651	3603	3554	3505	3455	3405
35	4218	4192	4166	4140	4113	4076	4038	3999	3960	3920	3873	3825	3776	3727	3677	3627	3576
36	4407	4380	4353	4326	4298	4260	4221	4182	4142	4101	4053	4004	3954	3904	3853	3802	3751
37	4600	4572	4544	4516	4487	4448	4408	4368	4327	4285	4244	4193	4141	4089	4036	3983	3930
38	4797	4768	4739	4709	4679	4639	4598	4556	4514	4471	4428	4384	4340	4295	4250	4205	4159
39	4998	4968	4938	4907	4876	4835	4793	4750	4707	4663	4619	4574	4528	4482	4436	4389	4342
40	5203	5172	5141	5110	5078	5036	5000	4961	4921	4879	4836	4791	4745	4698	4651	4603	4555
41	5412	5380	5348	5315	5282	5239	5195	5150	5104	5057	5010	4962	4914	4865	4816	4766	4716
42	5625	5592	5559	5525	5491	5447	5402	5355	5307	5257	5205	5152	5098	5043	4987	4930	4872
43	5842	5808	5774	5739	5704	5659	5613	5566	5518	5469	5419	5368	5316	5262	5207	5150	5092
44	6063	6028	5993	5957	5921	5875	5828	5780	5732	5683	5634	5583	5531	5479	5426	5373	5319
45	6288	6252	6216	6179	6142	6095	6047	5998	5949	5899	5849	5798	5746	5693	5639	5585	5530
46	6517	6480	6443	6405	6367	6319	6270	6220	6170	6119	6067	6015	5962	5908	5853	5798	5742
47	6750	6712	6674	6635	6596	6547	6497	6446	6394	6342	6289	6235	6181	6126	6071	6015	5958
48	6987	6948	6909	6869	6829	6779	6728	6676	6624	6571	6517	6463	6408	6352	6296	6239	6181
49	7228	7188	7148	7107	7066	7015	6963	6910	6857	6803	6748	6693	6637	6580	6523	6465	6407
50	7473	7432	7391	7349	7307	7255	7202	7149	7095	7041	6986	6930	6874	6817	6759	6701	6642
51	7722	7680	7638	7595	7552	7499	7445	7391	7336	7280	7224	7167	7110	7052	6994	6935	6876
52	7975	7932	7889	7845	7801	7747	7692	7637	7581	7524	7467	7409	7351	7292	7233	7174	7114
53	8232	8188	8144	8099	8054	8000	7944	7888	7831	7774	7716	7657	7598	7538	7478	7418	7357
54	8493	8448	8403	8357	8311	8255	8198	8141	8083	8025	7966	7907	7847	7787	7726	7665	7604
55	8758	8712	8666	8619	8572	8515	8457	8399	8340	8281	8221	8161	8100	8039	7977	7915	7853
56	9027	8980	8933	8885	8837	8779	8720	8661	8601	8541	8480	8419	8357	8295	8232	8169	8106
57	9299	9251	9203	9154	9105	9046	8986	8925	8864	8803	8741	8679	8616	8553	8489	8425	8361
58	9574	9525	9476	9426	9376	9315	9254	9192	9129	9066	9003	8939	8875	8810	8745	8680	8614
59	9853	9803	9753	9702	9651	9589	9526	9463	9399	9335	9270	9205	9139	9073	9006	8939	8871
60	10136	10085	10034	9982	9930	9867	9803	9738	9673	9607	9541	9474	9406	9338	9269	9200	9131

ONE END HINGED†																	
Square of least radius of gyration of cross-section (r^2) =																	
l	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2	d^2
d	6	8	10	12	14	16	18	20	24	28	32	36	40	44	48	52	56
12	962	954	943	933	921	908	894	874	846	816	784	748	708	664	616	564	508
13	1057	1047	1035	1022	1009	994	978	957	927	895	861	823	781	735	685	632	575
14	1157	1146	1133	1119	1104	1088	1070	1047	1015	980	943	904	861	813	762	708	650
15	1262	1250	1236	1221	1205	1188	1169	1145	1111	1074	1035	994	950	902	850	795	737
16	1372	1359	1344	1328	1311	1293	1274	1249	1213	1174	1133	1090	1045	996	943	887	828
17	1487	1473	1457	1440	1422	1403	1383	1357	1319	1278	1235	1190	1143	1093	1040	983	924
18	1607	1592	1575	1557	1538	1518	1497	1470	1430	1387	1342	1295	1247	1196	1143	1085	1025
19	1732	1716	1698	1679	1659	1638	1616	1588	1546	1502	1455	1406	1356	1303	1248	1190	1130
20	1862	1845	1826	1806	1785	1763	1740	1711	1668	1622	1574	1524	1473	1419	1363	1305	1245
21	1997	1979	1959	1938	1916	1893	1869	1839	1794	1746	1696	1644	1591	1536	1479	1420	1360
22	2137	2118	2097	2075	2052	2028	2003	1972	1925	1876	1824	1770	1715	1658	1600	1540	1479
23	2282	2262	2240	2217	2193	2168	2142	2110	2061	2011	1957	1901	1844	1786	1727	1667	1605
24	2432	2411	2388	2364	2339	2313	2286	2253	2193	2142	2086	2028	1969	1909	1848	1787	1724
25	2587	2565	2541	2516	2490	2463	2435	2401	2340	2288	2229	2169	2108	2046	1984	1921	1857
26	2747	2724	2699	2673	2646	2618	2589	2554	2492	2439	2379	2318	2256	2193	2129	2064	2000
27	2912	2888	2862	2835	2807	2778	2748	2712	2650	2596	2535	2473	2410	2345	2279	2213	2148
28	3082	3057	3030	3002	2973	2943	2912	2875	2812	2757	2695	2634	2571	2507	2442	2376	2310
29	3257	3230	3202	3173	3143	3112	3079	3045	2980	2924	2861	2798	2734	2669	2603	2537	2470
30	3437	3409	3380	3350	3319	3287	3253	3218	3151	3094	3030	2965	2900	2834	2767	2700	2632
31	3622	3593	3563	3532	3500	3467	3432	3396	3328	3270	3204	3137	3070	3002	2934	2865	2796
32	3812	3782	3751	3719	3686	3652	3616	3579	3510	3451	3384	3316	3247	3178	3108		

Mr. Seaman. HENRY B. SEAMAN, M. Am. Soc. C. E.—It was not the purpose of the paper to offer anything strikingly novel, or unusual, but rather to review the experiments upon which the fatigue formulas are based, and after making reasonable deductions from them, to outline a specification which would embody these results in a concise and practical form. It is to be regretted that any one should have misconstrued these deductions as advocating any precise or restricted interpretation of the tests, as it was repeatedly noted that they were too few, and the material too variable, to permit of any such construction. To propose a new formula, even of the form outlined by Professor Marburg, $C = [\text{Min.} + 2 (\text{Max.} - \text{Min.})]$ would be to imply a definite knowledge which the experiments do not warrant. At best, the 2 to 1 ratio for fatigue is an approximation; but, being so considered, it is a better general interpretation of those results than is any precise formula which can be presented.

The use of the Launhardt formula has long been considered the most refined interpretation of scientific investigation, and has been extensively introduced in general practice. When the review of the experiments upon which this formula was based showed its inconsistency with those tests, and a more careful analysis tended to confirm what had been the best practice of former years, it seemed proper that the matter should be laid before the Society for consideration. One of the strongest endorsements which any method of dimensioning can receive is the fact that its general features have been long in vogue, and if, in addition to this prolonged usage, it may be shown that the most accurate and scientific experiments yet made upon the resistance of materials, serve generally to endorse this same practice, it will be an additional confirmation which should serve to establish the system.

That the Launhardt formula should have received so little defence in the discussion would appear strange, were it not for the fact that no defence can be made. Mr. Wilson's request for further experiments before discarding the formula would indicate that he has not fully reviewed those already presented. The results of these tests alone, destroy the foundation upon which the formula rests, and its utter unreliability is most fully appreciated after reviewing the method of derivation.

The table comparing the results of the formula with those of experiment, as published in the translation quoted, was as follows (in German centners per square inch*):

	0	250	400	600	1 100
Experiment.....	500	700	800	900	1 100
Formula.....	500	711	800	900	1 100

It will be seen that in four cases out of five the results agree exactly, and it is this presentation which probably misled Mr. Wilson, and

* To reduce to pounds per square inch, as per *Engineering*, multiply by 107.

many others, to adopt the formula. Since, however, the value of 1 100 Mr. Seaman. is not in Wöhler's tables of duration tests for spring steel, it would be interesting to know by what higher authority it was introduced by Weyrauch. It is also important to know whether 1 100 centners represents the weight at which the material broke, or the maximum load under which it did not break. In either case, the length of time the test piece remained under strain, as compared with the other duration tests, is important. It is evident that if sufficient time is allowed to receive the advantage of judicious cold-drawing, the ultimate strength will be materially increased. The table, corrected and reduced to English equivalents, is shown on page 147.

From an inspection of Set No. 1 of spring steel it is apparent that the value 53 500 lbs. (500 centners) is too low, and that a weight between this and the next higher (64 200 lbs. or 600 centners) would have been sustained indefinitely.

A change in either the value of u (500 centners) or that of t (1 100 centners) would be sufficient to change the results of the Launhardt formula. Since one of these values is undoubtedly erroneous, while the other is very questionable, there seems little excuse for its further consideration. The fact that u (500 centners) is less than one-half t (1 100 centners) is a further indication of abnormal conditions.

The confirmation of this formula would therefore appear very unsatisfactory, but the manner in which the formula was derived is even less convincing. The statement that the working strength is a function of the live strain is the most vague and indefinite conceivable; and to satisfy this and establish the formula by the interpolation of an arbitrary value which will fill certain stated conditions might suggest inductive rather than deductive reasoning.

Although the Launhardt formula fails, the theory of fatigue has met with more general acceptance than was anticipated. The chief objection appears to be, as already suggested, to the use of the word, rather than to the acceptance of the idea. Mr. Cooper, whom the author regarded as the most prominent opponent to this theory, suggests that within a certain "range of action" (which the author would term range of work, "power of work," or endurance), the elastic capabilities of the material may be measured in cycles of strain, and he includes in this cycle the sum of alternate strains. This is precisely the idea which the author intended to convey, although he did not adopt the word "elastic" because it had been formerly used with a different interpretation.

Mr. Schneider more ingeniously adopts directly the definition of the yield point, or Bauschinger's limit of uniform elongation for his new definition of "elastic limit," overlooking the fact that the original theory of perfect elasticity required not only that the material elongate in direct proportion to the amount of applied strain, but also

Mr. Seaman. that it recover completely its original unstrained condition as soon as the applied load is removed.

It is due to the fact that "after rest," as Mr. Cooper has happily expressed it, the material returns "to its original form," that the term fatigue has been used. It is analogous to the condition of animate nature, which is capable of doing a certain amount of work, provided its range of action is not exceeded; and even exceeding this range or limit, it will recover, after rest. The instance of the fishing snells, mentioned by Mr. Waddell, illustrates its general acceptance, and barbers bear similar testimony in regard to their razors.

Professor Johnson's theory that the true explanation of fracture is due to the development of micro-flaws rather than to fatigue might be tenable if the experiments had all been made on homogeneous steel and the results had proved less uniform; but since fibrous wrought iron was used, in which micro-flaws do not develop, and since the results of the tests on spring steel are comparatively uniform, while the development of micro-flaws is generally confined to a high sulphur steel, is more rapid in hard than in soft material, and does not show any uniformity in results, the micro-flaw theory does not appear to be sustained by experiment.

Soon after the publication of the paper, a statement appeared in one of the technical journals, that since the allowable unit strains are within the elastic limit, there is no necessity of considering fatigue. Several discussions have followed the same suggestion. Such a statement violates the fundamental principle of scientific bridge construction. A well-designed bridge must be as uniformly economical as unlimited life and absolute safety will permit, and there is no more reason why the dead strain should be a ratio of the elastic strength than that the live strain should be a ratio of the alternating strength. It is not merely necessary to build a safe bridge. Such a bridge may be constructed by the selection of a uniform allowable strain based solely upon the alternating strength, but it would be a design which no engineer would endorse.

It is true that bridges are not built to fail, but the proper design of bridges has the point of failure constantly in view, not merely for the purpose of avoiding it, but for the purpose of uniform economy throughout. A factor of safety is selected to cover contingencies which may exist. Where the strains are defined and the contingencies are possible defects in material and workmanship, this factor is usually 3, and provides that only one-third of the full area of the member may be relied upon as perfect when compared with the results of a test specimen. While, therefore, the material may receive a nominal strain of 18 000 lbs. or more per square inch, estimated on the total section, it is supposed actually to be strained close to the ultimate strength of some unseen imperfection. It is this critical

section, always assumed and ever possible, to which the bridge is Mr. Seaman. designed. This is economical bridge construction, and anything short of it is a crude approximation, wastefully expensive in proportion as the light of experiment and reason is hidden, and the impulse of arbitrary and unreasonable precaution or *dictum* is followed. If the factor of safety is assumed too large it should be reduced, but advantage should not be taken of it to confuse other irregularities which may be properly separated and defined.

Furthermore, it is not merely the construction of bridges which must be considered in outlining specifications, but also their maintenance, since it is here that the closest study of existing conditions is required and for the lack of which many efficient bridges have been consigned to the scrap heap. When the overloaded structures of im-

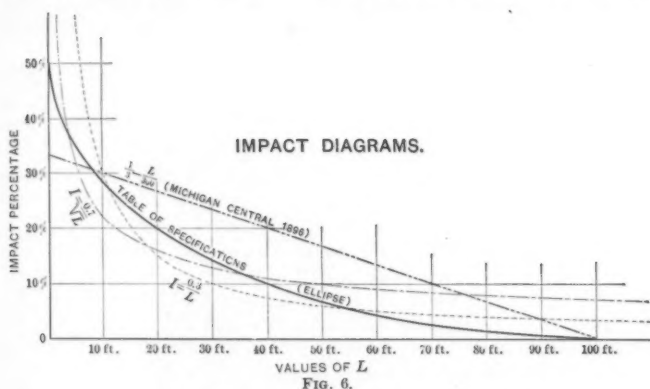


FIG. 6.

poverished roads are to be maintained to the extreme limit of absolute safety, it requires a most careful consideration of the capabilities of the material. The experience of Mr. Fowler in deciding upon the final life of a bridge but illustrates the responsibilities of many others in similar positions. It is a very easy matter to condemn a bridge, but not so easy to maintain it to the final limit. In such cases the question of fatigue is an important one.

The statement that a table for impact complicates an otherwise simple method of dimensioning is, unfortunately, true, but unavoidable. Several years ago, after reading the paper on "Stresses in Bridges,"* by Wm. H. Booth, M. Am. Soc. C. E., the author undertook the theoretical investigation of the effect of impact, upon the assumption that a load instantaneously applied will produce twice the

* Transactions, Am. Soc. C. E., Vol. xx, p. 137.

Mr. Seaman. effect of the same load at rest; also that the effect varied inversely as the time consumed in its application. The formula

$$I = \frac{\text{Constant} \sqrt{L}}{L} = \frac{\text{Constant}}{\sqrt{L}}$$

was adopted as a probable representation of the effect of impact. When the present paper was written, the notes upon which this formula was based had been lost, and the formula was, therefore, replaced by the table, the values of which will be found to approximate the curve of an ellipse, as shown in the diagram, Fig. 6.

The formula indicates that impact will be much increased for very short spans and for first panels of long spans. The diagrams of impact taken, both by Professor Robinson and by Professor Turneaure,* confirm this fact as for long spans, and Professor Turneaure's statement that the impact for short spans was so sudden that the instrument would not record the results correctly appears to give confirmation for short spans as well.

The effect of oscillation after the train is fairly on the bridge is given by Professor Robinson† at 50% and by Professor Turneaure as 20 to 30%, with one test on Bridge No. 13 showing 33 per cent. Professor Turneaure also states that the total impact (including vibration) on spans of 50 ft. and less, may be taken at 40 to 50%, and decreases rapidly to, say, 25% for 75-ft. spans, and owing to cumulative effect will remain approximately constant for longer spans (to 150 ft. or more). By adding Professor Turneaure's constant for oscillation (say 30%) to the values of the table, it will be found to give 50% for 20 ft., 40% for 40 ft., 37% for 50 ft., and about 30% for 75 ft. or over. Although impact experiments are extremely rough and approximate, the confirmation is of especial interest. It will be interesting to note what subsequent experiments may show on spans under 20 ft. in length.

The statement that the present specifications were written from the standpoint of the railroad company, rather than from that of the manufacturer, has elicited criticism, but it is not clear upon what such criticism is based. A bridge engineer, with training in bridge shops and experience on maintenance, should be able to weigh the cost and value of various shop requirements and to specify what is most economical to the railroad company, as the purchaser and user, rather than to the manufacturer as the seller. An instance in point is the chemical analysis. The manufacturer's specifications permit 0.08 phosphorus for basic and 0.10 phosphorus for acid steel. The requirements of 0.04 phosphorus for basic is almost universal with railroads and may be obtained without extra cost.

The question of replacing the typical engine of specifications by an equivalent uniform load, with a single concentration, is one which

* *Proceedings*, Am. Soc. C. E., Vol. xxiv, p. 783.

† *Transactions*, Am. Soc. C. E., Vol. xvi, p. 63.

has already been considered* and the reason for not doing so now is, Mr. Seaman. that such a radical change in a general specification should be the work of a committee, if one should be appointed, rather than that of an individual. The proposition appears to be a practical one, and has been adopted by some roads,† but there is a strongly conservative feeling that where the typical engine differs from the equivalent load, the latter conforms more closely to actual conditions, and should be retained. The author believes the difference is so small that the change would be acceptable for new work on account of the greater simplicity.

The reference of Mr. Snow to the importance of making structures rigid as well as strong deserves the greatest emphasis. It is this feature which makes the plate girder the best railroad structure in service, the riveted lattice truss next, and reserves pin-connected work for long spans where the dead load is sufficiently large in proportion to the live to give the structure rigidity. The dead, or initial, strain in any member should be sufficient to overcome any recoil from impact or vibration of the moving load, and where the dead strain is not sufficient, provision should be made for a rigid member or for an initial strain produced by counters. The latter method was successfully used in the Post trusses which were often of short spans, with counter rods in every panel. They were apparently as rigid as the lattice truss, and if the same methods were applied to short Pratt trusses with rigid lower chords, they would probably be found quite as serviceable.

The statement that webs of plate girders resist bending strains is undoubtedly true, and it is partially due to this fact that the plate girder gives such good service. Properly designed web plates which are bulged, however, were probably made so in manufacture, and the suggestion that webs be made sufficiently strong for collision is one provided for by the stiffeners, and which no specification can cover. Although the plate girder is not an articulated structure, the use of stiffeners makes it approximately so, as described by Rankine, the web acting both in tension and compression, the former predominating and the latter existing only to such extent as the tensile strains may stay the web while acting as a column in the diagonally opposite direction. Instances have come to the author's notice where the vertical strain through the stiffener fractured the support, while the strain from the adjacent web, which on account of the flexure of the girder should have been first to act, produced no noticeable effect. Some specifications permit one-sixth of the web section to be estimated as flange area, and it will do this service if it is continuous, but it is not an easy matter to properly splice the web for bending, and with rivets in the vertical splice spaced three diameters apart, it leaves but $\frac{2}{3}$ of

* Transactions, Am. Soc. C. E., Vol. xxvi, p. 229.

† Specifications, Pennsylvania Lines West of Pittsburgh.

Mr. Seaman. the web, or $\frac{2}{3} \times \frac{1}{3} = \frac{1}{3}$ of the total section available for equivalent flange section. The clause of the specifications considering the web as resisting shear only, is in very general use and is a conservative provision. In the review of old structures, however, it is proper and customary to make allowance for the resistance of the web.

The placing of stiffeners of plate girders has always been subject to arbitrary specification, and that used in the present instance appears to be the common practice. The suggestion as to double stiffeners at points of support is good, and is not ruled out by the specifications. On the other hand it would be required, whenever necessary, to properly distribute the strain. The proposition to complicate the spacing of stiffeners according to a formula does not seem necessary or advisable.

The explanation of the different formulas in use for finding the allowable strains of rollers, $c \times \sqrt{d}$ and $c \times d$, is very clearly presented by Mr. Morison. The former is based upon the resistance of materials to crushing; the latter is based upon the resistance to the motion of rolling, and is advocated with the statement that the material of bed plates might safely be cold-rolled. This could hardly be accepted to the unlimited degree the formula would permit, and, if carried sufficiently far, would result in the shearing of rivets and general distortion of the metal. Its application seems only justified as an encouragement to the use of large rollers, which feature may be otherwise specified. It would seem preferable, therefore, to adhere to the old form $c \times \sqrt{d}$ and use different values of the constant c for dead and for live load. The latter provision would be for uniformity in the specification rather than for any resulting effect of these loads, since the strains are not received consecutively by the same sections, as the roller moves.

The proposition to specify different allowable strains for different kinds of masonry, such as brick, concrete, etc., implies that this masonry may support the iron bed plate. Such would not be good practice, even for highway bridge work. In every case a proper stone pedestal block or coping is furnished, which will sustain and distribute the applied load of 300 lbs. per square inch. Specifications for the metallic structure should not extend beyond this block.

The proposition to limit the application of the column formula comes from those who have used the straight-line formula, which is of empirical origin and applicable only between limits. The rational formula, however, is not governed by the same limits, but is applicable to long columns. The suggestion that the ends of a riveted lattice web member are fixed is hardly consistent with the statement that the top chords should be considered hinged. If a top chord is hinged, the panel point becomes one of contra-flexure, and this fact would give an initial moment to the web member which would be more injurious

to a column under compression than if the connection were not Mr. Seaman riveted.

The clause forbidding alternate strains has been very justly criticised, and should be restricted to pin-connected members as originally intended.

It is true that the usual specifications for sliding friction provide for a coefficient of 20%, and this may be obtained under the driving wheels, but the usual friction of stopping trains will not exceed 10%, and the most perfect brake tests on long trains, which have come to the author's notice, showed a coefficient of less than 13 per cent. Since, however, the allowable strain has been raised, it might be advisable to retain the 20% for uniformity. For the same reason the wind pressure should be increased to 40 lbs. to conform to the general practice of building construction. There appears no reason for the distinction which has heretofore existed between these classes of structures, in this respect.

The clause requiring upper-flange plates on all plate girders is chiefly one of aesthetic finish, though it also serves to cover the small water pocket formed in the flange by the web plate and angles. Where there is no upper-flange plate, the web may be made full and no pocket formed, and for deck girders it is decidedly preferable to avoid the varying thickness of flange plates, as well as the rivet heads. It would therefore seem preferable to avoid upper-flange plates entirely, wherever ties rest upon the upper flange; but if any cover plate is used, at least one should extend full length, and all through girders should be so finished. It is not uncommon also to specify round corners on through girders, but as these are particularly difficult to construct, such a clause may be left to the individual preference of the engineer.

The objection to open turnbuckles is principally that they are readily tampered with by the use of a bar. There are few roads which would permit a switch to remain unlocked, and a bridge member would seem almost as important. The author has heard of instances where sleeve-nuts had the lower thread tight, the upper thread loose, became filled with water, froze, and cracked, but has been so unfortunate as never to have seen such an instance. He has always wondered why the lower thread should always remain tight, why it did not become loose after the water froze the first time, and why the material did not stretch a little before cracking. Neither has he been able to understand why a sleeve-nut with tight, compact thread, but with body split by frosty nature, should not be as good as an open turnbuckle, with sides opened in manufacture. The ends of the rods may be located by the punch marks, 12 ins. from the end of each bar, and, if deemed necessary, drain holes may be drilled in the nut.

Mr. Seaman. There does not appear to be unanimity as to whether or not there should be tension on rivets, some preferring no tension, while others claim they will stand full tensile strain. Both are in a measure right. If the rivet holes are perfectly reamed, the edges rounded to $\frac{1}{8}$ in., and rivet heads sufficiently large, there seems to be little reason why a steel rivet should not stand full tension; but when we consider the difficulty of obtaining work of this character, the danger of its neglect, even when specially required, and the impossibility of detecting its omission in the finished work, the use of rivets in tension is a very questionable practice, even though carefully-made tests show favorable results. It seems preferable, therefore, to avoid tension on rivets wherever possible, and in no case to allow more than one-half value.

The suggestion that inspectors should give detailed reports of condemnation, etc., is of very doubtful wisdom. Such a report is interesting and appears to indicate that the inspector's services are valuable, but the custom is very apt to lead to inflated reports and to endless conflict in the shops. The report of an inspection firm which recently came to the author's notice was so full of condemnations and corrections of bad work, that the structure should have been condemned as a whole, and the shops closed. In order to avoid just such artificial reports, the author provides that memoranda may be kept, but no such report made unless called for. By this method, temptation to inflate is avoided, and the engineer receives much closer co-operation in the shops. Only men who are known to be true should be employed, and their work should be checked. In this way a much higher class of work is secured with little friction. It is for the same reason that the responsibility is thrown upon the contractor until the work is completed. The result obtained is the criterion sought.

Mr. Breithaupt's explanation of the distinctive use of the words "stress" and "strain" should be carefully read by all who wish to adopt the former term. The indiscriminate interchange of these words has become very general, even by the best authorities, and it must try the nerves of those who adhere to the distinction, to see such statements as "the material was stressed beyond the elastic limit." The author's preference, however, is to use the word strain in every case. A stress is a delicate force, or tendency to force, which has no place in railroad bridge construction. It should never be used outside of the class-room, and should be restricted there as far as possible. The distinction between applied strain, and internal or molecular strain, may be made when necessary by the proper use of the adjective, and even Rankine, when he suggested the distinction, could hardly have foreseen that incongruous combination "stress sheet," as replacing the good old "strain sheet" of former times.

The question suggested by Professor Merriman as to whether the

percentage of impact be added to the load or to the strain may be Mr. Seaman. answered by considering the purpose of such impact allowance, which is to provide for the increase of applied load due to sudden application. This is directly proportional to the live strain in every instance, even in the case of counter strain, for the live strain is not merely the alternate strain, but it is the total live strain which first neutralizes the initial dead strain, and then, still increasing, produces the alternate strain in the opposite direction.

If Professor Winkler, in the face of Wöhler's experiments, endeavored to disprove the results shown by them, it is not surprising that his task has been a difficult one. If, however, it is their abuse which he deprecates, whether it be an unwarranted precision in their interpretation, or an entire disregard of their teachings, he has led where others must follow.

There is no greater abuse of these experiments than their deliberate misinterpretation, or an endeavor to disparage their results. Precise deductions are not warranted, but to limit their teachings to the statement that there is no fatigue within the elastic limit is to beg the question entirely. To state, as Mr. Schneider does, that "the elastic limit is actually the ultimate strength," is to affirm what these experiments directly disprove. If the "elastic limit" of 34 240 lbs. is the real ultimate strength of fibrous iron, how does the material stand a strain, varying between 47 080 lbs. and 25 680 lbs., repeated indefinitely, without fracture? If the "elastic limit" of 53 500 lbs. is the real ultimate strength of spring steel, how does the material stand, without fracture, strains varying between 74 900 lbs. and 26 750 lbs., between 85 600 lbs. and 42 800 lbs., or between 96 300 lbs. and 64 200 lbs., each in turn repeated indefinitely? The statement "that a single strain (beyond the elastic limit) has practically destroyed the usefulness of the material, and will ultimately produce rupture, if repeated often enough" is absolutely without foundation and contrary to established results.

Table No. 1 as published by Mr. Schneider is interesting, but would have been more valuable if he had described in detail the method and rapidity of loading, so that the difference between these results and those of Wöhler, or those of Mr. James Howard cited by Professor Merriman, might be explained.

The question of impact has already been reviewed, but when Mr. Schneider states that his specifications of the Pencoyd Iron Works for 1887 make a distinction between live and dead strains, he overlooks his former claims that he allowed the same unit strains for live and dead loads, merely increasing the former for impact by formula. He is correct when he states that his provision for impact is far in excess of that shown by experiment. Does not his impact formula also cover fatigue? The author has so regarded it.

Mr. Seaman. Although wind is live strain, it rarely occurs, and the allowable dead strain is permitted. To allow the same strain for a force which is rarely, if ever, applied, as that allowed for one which may be applied at every loading, would be to lose sight of the practical application of the results.

The present tendency toward riveted-lattice trusses for moderate spans is commendable, for the reason that provision is made for reaction after sudden application of the live load, when the live load is large in proportion to the initial dead strain in the member, though there are always objectionable secondary strains. The extent, however, to which the practice of rigid bracing is sometimes carried by manufacturers is ludicrous. It is not an uncommon sight to see the diagonals of trestle bents, or the main laterals of truss bridges, made of angles so long that they sag of their own weight, are of no value as struts, and permit no initial counter strain to give rigidity to the structure.

The reciprocal of the column formula $\left[P = p \left(1 + \frac{l^2}{18\,000r^2} \right) \right]$ will, for alternate strains, replace that formula and not be used in addition to it. This method of dimensioning, therefore, is as simple and direct as that of adding the sections, and is more clearly rational.

The clause requiring six rows of rivets in flanges of plate girders over 16 ins. wide is superfluous and may be omitted. The provision is better covered in a general clause.

The specifications do not require stiffeners on plate girders 24 ins. deep to be spaced closer than 4 ft., unless necessary for concentrated loading.

Terminal cross-bracing plates are properly called batten plates. Tie plates on a railroad are used to prevent rails from cutting into the ties. It is a common practice to make batten plates $1\frac{1}{2}$ times the width of the member.

The first and third clauses by which Mr. Schneider proposes to avoid "gim-crack" construction are good, but if the second clause is used, it should read "resist compression as well as tension," since all lateral systems resist tension, and but few resist any appreciable compression. It should also specify the amount of compression which must be sustained.

The criticism of "lop-sided" unsymmetrical construction will be freely endorsed, and general practice has long avoided it wherever possible, but the plea that bending moments cannot be avoided by placing the pin in the neutral axis is not altogether clear, since the distance from the center of the pin to the center of gravity of the section is a factor of that moment, and reducing this to zero will eliminate any result.

The clause requiring parts to be assembled in the shop is usually

executed by template reaming and should be modified, as Mr. Mr. Seaman, Schneider proposes.

Eye-bars, properly manufactured of soft steel, will not require annealing.

While it may be true in a restricted sense, as Mr. Morison suggests, that columns subject to compression fail by tension, it is not true in the broader interpretation of the condition. A column placed under compression in a testing machine has no other strain applied, and any buckling or apparently tensile fracture may be as truly the result of compression as of tension. There can be no tension developed in any fiber except by a corresponding increase of compression in other fibers, and this increase of compression must be added to that which is received from the end thrust. It is therefore probable that there must be incipient failure by passing the elastic strength in compression, before the buckling takes place, and this, in fact, would cause the buckling. The failure of the column, therefore, is solely and entirely due to compression.

The purpose of allowing different dead and live strains, is three-fold: 1st, to provide for any possible effect of fatigue; 2d, to provide for any augmentation of the live load due to vibration; 3d, to provide, to some extent, for possible future increase of moving load over that assumed in calculation. In the first instance it is generally understood that fatigue exists under compression as well as under tension, and requires that alternate strains be added. Also that when the metal is overstrained in one direction, it loses at once its power of resistance in the opposite direction. In the second instance the increased strain due to vibration, and provided for, even by Mr. Morison, in the tension members, is transmitted directly to every compression member. In the third instance any possible future increase of moving load affects every member of the structure, whether in tension or compression. There is, therefore, no reason why a uniform strain should be adopted for compression members while separate strains are used in tension members.

In the statement that plate girders should be made as nearly a solid piece as possible, the author would concur, but he does not share the doubt expressed as to the efficiency of the outer flange plates. No rivets of greater length than five times their diameter should be driven, and holes should be reamed perfect, before driving. If these rivets fill the holes solid there is no reason why they should not transmit strain as well as an equivalent amount of solid material. The practicability of this feature is in the workmanship, and this, in turn, depends upon the specifications and the shop inspection. The greatest increment of strain transmitted to the flange plates occurs at the ends where the thickness of these plates is least. Any strain transmitted near the end is thoroughly distributed over the entire pile by

Mr. Seaman. the time it reaches the center, and the increment at the center of the girder is readily provided for by the large excess of rivets which are always there present.

It is somewhat disappointing to find less corroboration in the Michigan Central specifications than was anticipated from Mr. Schaub's remarks. Those specifications allow live strains at two-thirds the allowable dead strain, and use the impact formula $I = 1\frac{1}{2} - \frac{L}{300}$ if L is less than 100 ft. Other than this there seems to be no confirmation.

The practical application of the 2 to 1 ratio far antedates 1882, as has been so fully cited by Mr. Wilson; but that fact does not detract from the value of Mr. Schaub's endorsement.

Mr. Wright deplors the prospect that machinery, now used for reaming high steel, may become idle upon the advent of the softer material, but it is doubtful if such would be the case. Reaming has been specified for the purpose of removing injured material around the holes, but the matching is often so imperfect that all reaming is on one side of the hole, leaving the other side untouched. Under these circumstances the end sought is not accomplished and a softer steel is advocated. The mismatched holes continue, however, and it is probable that these machines, now used to remove injured material will ultimately be required in all cases to produce perfect workmanship.

The author regrets to have been misunderstood by Mr. Worcester as advocating the use of the Launhardt, or of any other precise fatigue formula. The purpose of the paper was to show that no exact formula was justified by the experiments, but rather that fatigue could be considered in a more general, though only approximate, ratio.

To the inquiry as to whether the same allowance be made for impact in floor beams of a double-track bridge, as would be provided for a single-track, the reply would be the same as for other parts of the structure. If it is possible to load them simultaneously it may also be possible to receive the same impact simultaneously.

The proposition to abandon the intermediate transverse bracing in double-track through bridges has been often urged, but the objections to its use appear to the author unfounded, and the bracing is advantageous in giving compactness to the structure. The side deflection described is inappreciable, and decreases the uneven vertical deflection, thus tending to keep the floor level under partial loading. The increased deflection of one truss beyond the other, which results from the omission of this bracing, tends to increase rather than to decrease any uneven deflection of track. The arguments offered for its omission apply equally well to high through bridges, with overhead sway

frames. Is it also proposed to omit these, and intermediate knee Mr. Seaman. braces as well?

The reason for the omission of the lower limit in the ultimate strength of rivet steel, is that no material which will show the elongation specified can fail to show a satisfactory ultimate strength. There is no objection, however, to its insertion, if preferred.

To the suggestion that oil be used instead of paint for surfaces in contact, because the practice is to paint but one surface, it may be said, that both surfaces should be painted. This is a question of inspection. Oil without pigment is not a permanent protection to iron.

The clause for initial strain in adjustable members, quoted from Mr. Snow, is most excellent, though to those not familiar with its use it might appear to elaborate a somewhat arbitrary requirement.

In Professor Turneure's suggestion that the specification does not make sufficient provision for impact, he has apparently overlooked the fact that the ratio of 2 to 1, as used in the specification, provides for vibration, etc., as well as for fatigue. What proportion of this ratio is really necessary to provide for fatigue may be a debatable question, but if Professor Turneure's allowance of 30 to 40% be taken and the difference—say, 60%—be added to the quantities given in the impact table, the allowance will be found to be very liberal.

The suggestion that centrifugal force be treated as a live load, is pertinent, though it should be remembered that this force is entirely dependent upon the speed, which is variable and uncertain, while the vertical loads are dependent upon gravity, which is always present. The bridge being designed for maximum conditions, and the item a small though important one, it would be well to make the provision suggested. Sixty feet per second is a maximum speed for heavy freight trains, for which the bridge is designed.

Of the general importance of familiarity with shop practice by designing engineers there can be no question, but to the attitude of Mr. Moulton that, as compared with engineers employed by railroads on maintenance, shop specialists alone understand the science of bridge design, the author would take most strenuous exception. Experience in shops, contemporary at least in part with that of Mr. Moulton, has taught the author that the best designs which "manufacturing engineers" produce, are those which maintenance has shown to be most serviceable, and which will be found in every case, to conform to some sound principle of mechanics. The statement that manufacturers "know their specialty thoroughly" is reassuring, and if true, there should be no necessity of the shop inspector assuming the functions of a consulting engineer as Mr. Moulton suggests. The assumption that the manufacturing engineer has sufficient knowledge for the inspector's guidance is the very danger to be avoided. It might save the reference

Mr. Seaman. of important questions to headquarters, but could hardly be considered advantageous from the railroad standpoint.

The economies of the past few years are not due to any change in design to conform to particular shop practice, but rather to the financial stringency through which the country has passed during that time. Prices were reduced so low as barely to pay running expenses, in order that bridge companies might continue to exist. It has resulted in cheaper work, but the work has also deteriorated in quality, and it is believed that renewed activity will, to some extent, restore former conditions.

The statement that a specification "should be broad enough to allow the special knowledge of manufacturing engineers to be available," explains why it should embody general principles rather than specific details.

Bending and drifting tests have been omitted because they are crude in comparison with the tests specified, and are valueless either to confirm or to refute those tests. It is very poor material which will not stand the drifting test, carefully made, and such a test is worthless in accepting material, though it may reject it.

The specification for wrought iron is for high-test material to be used where loop-eyes or screw-ends are desired. It is believed that iron is preferable in all such cases, even with the same unit strain, to the best grade of steel.

Square corners in re-entrant angle cuts should be avoided, as Mr. Moulton proposes.

The clause allowing three-fourths value for oblique section between rivet holes is based upon the decreased power of an oblique resistance, rather than upon any similarity to fibrous wrought iron.

The suggestion of Professor Ricketts that rest "equal to the length of time between trains on a railroad bridge" may remove any effect of strain, depends upon the frequency with which trains are run. In this connection the table of Mr. Breithaupt showing that material will stand a greater number of repetitions at the rate of 72 per minute than at the rate of 18 per minute is interesting. If it takes days to remove the effect of strain, the use of the bridge must be greatly restricted. The author has already expressed his concurrence with the view that precise fatigue formulas are not justified, but he believes that fatigue must receive consideration in a structure which may be subject to constant service.

Mr. Fowler's method of deducing safe strains can hardly be considered an independent demonstration. It is applied in the extreme values only, and, therefore, is no more a confirmation of the Launhardt formula than it is of the 2 to 1 ratio. The method which is sometimes used in reviewing old bridges is to assume an extreme value for u in the Launhardt formula and find the corresponding strain allowable.

The contention of Mr. Snow that Wöhler's experiments are too few Mr. Seaman. to justify any precise formula for fatigue is well founded. They do show that fatigue exists, but the exact effect can only be approximated, because the element of time is an important factor and has not received sufficient consideration in the experiments.

The question whether web members of riveted lattice trusses should be considered fixed or hinged has already been discussed, but the suggestion that upper chords of lattice bridges may be partially fixed while those of pin trusses are certainly hinged, calls attention again to the reason for the distinction. The difference is due to rigidity which may be received from the web members, and this can only be produced by a severe bending strain, which would be more dangerous to web members under compression, than if their ends were free.

The use of high steel for eye-bars seems illusory because the bars are thoroughly annealed, and the material so softened that a milder product might have been specified in advance. It is a moot question whether annealing is not more injurious than beneficial to the material. Hard spots and incipient cracks are more prevalent in hard than in soft material and are especially objectionable under vibratory tensile strain.

The defects incident to high steel would appear to be more dangerous in pins than in any other part of the structure.

The question of fatigue and its application to bridge construction has already been considered in this discussion, but the following statement by Mr. Waddell illustrates the tenacity with which the term "elastic limit" is retained: "For stresses of opposite kind the sum of the two intensities, when rupture is produced, is also greater than the elastic limit." The fact that the alternate strains must be added, to equal the "elastic limit," destroys that term, as formerly accepted. If the material were perfectly elastic up to 34 240 lbs. per square inch, a strain of - 25 000 lbs. could be applied and removed without effect. A reverse strain of + 25 000 lbs. could likewise be applied and removed without effect, and the material could alternate indefinitely from one extreme to the other. Such is not the case, however, as the material stood only 17 120 lbs. under reverse strains. This demonstrates that 34 240 lbs. was not an "elastic limit" but, rather, was its range of action, or power of work, or endurance.

Mr. Waddell appears to be somewhat confused as to the distinction between the effect of fatigue on a 2 to 1 ratio, and the effect of a suddenly applied load which produces twice the strain of the same load at rest. The suggestion "that every live load is twice as destructive as a dead load" is startling, as it would thus require but four repetitions of the allowable live strain to destroy the material. Could the author have been so misunderstood? To the statement that "the live load of the experiment was just twice as effective as a dead load,

Mr. Seaman. in that it was applied suddenly," Professor Marburg's statement is a correct reply:

"It is to be remembered that Wöhler's tests were not made by the application and release of known loads, in which case the stresses would have varied with the rapidity of the operations, and would have been different above and below the elastic limit, with other conditions constant. The apparatus was, on the contrary, so adjusted by means of calibrated springs that the stresses themselves remained sensibly constant. This distinction is an exceedingly important one."

It is unnecessary to add that the Launhardt formula has not been introduced into the specification, as Mr. Waddell states.

The impact effect upon a panel suspender, as compared with that of a chord, has no connection with the question of fatigue.

"Unequal tearing" alludes to the practice of connecting but one leg of an angle in tension, and making no provision for the resulting moment, or tendency to tear, in the piece.

The author has not found opportunity to read "De Pontibus," but if there is anything therein contained which permits a variation in punching exceeding that for which the bridge is designed, it is bad practice, and should be corrected. If engineers are thus lax, what may be anticipated from inspectors?

To Mr. Cooper's discussion the author has little to add. If he prefers the expression "range of action" to "range of work" or "power of work," the preference will not be disputed, and even if the old term, "elastic limit," were completely destroyed in its old meaning and revived in a new one, there might be little objection, although there is no "limit" proper, but rather an elastic strength or power of work. The practical impossibility, however, of completely shaking off the old interpretation is shown by Mr. Cooper's suggestion that occasional tests be made for comparison between it and the "yield point," which is now found in practical testing by "drop of beam."

If the time should ever come when stone masonry is shown to have a definite strength within a certain "range of action," and to require a time of "rest" to restore its original strength, we may well use the expression, "fatigue of stone walls."

Mr. Giles' suggestion that the 2 to 1 ratio provides sufficiently for impact implies that impact would be the same for all spans. All experiments made, however, show conclusively that there is a large excess due to sudden application of the load on short spans and on the first panels of long spans. There appears to be no way of providing for this except by a special impact formula or table.

The clause providing that wind pressure, etc., shall not produce greater strain per square inch than that allowed for dead strain, is based upon the infrequency of the strain rather than upon any supposition that wind, etc., is dead load.

The author wishes to endorse Mr. Giles' suggestion that tension on

rivets should be avoided, but if in exceptional cases it is unavoidable, Mr. Seaman. a prescribed allowable strain should be established.

The clause providing for the use of bolts instead of pins refers to rough bolts, but they might well be omitted altogether on this class of work.

The suggestion of Mr. Greiner that a committee should be appointed for the purpose of procuring greater uniformity in bridge specifications deserves support. There is little danger that the work of such a committee would be misunderstood or considered a bar to individual development, but it would tend to eliminate many useless differences which at present exist, and would record individual views in discussion as no single paper can possibly do. The difficulty in the work of such a committee, however, is little realized by those who have not undertaken it. The Committee on Rails took four years in which to formulate a report on simple shapes, with few items in theory or practice to harmonize. The Committee on Tests of Material worked five years under similar conditions, and, after repeated unsuccessful attempts to have meetings, formulated a brief report by correspondence. It was with appreciation of the difficulties of such work that the present specifications were offered for discussion, and it is gratifying to note that those taking part have done so in the same spirit with which they were offered, and have not confined themselves to intangible generalities, but have made most valuable suggestions and modifications.

As has already been noted, railroad specifications should be written from the standpoint of the railroad company, and in so doing every economy will be considered, but it is the economy of durability rather than that of inferior workmanship.

It is to be regretted that anyone should hesitate to discuss matters of opinion. Specifications can hardly be written without them, and an opinion which will not bear discussion and possible modification is of doubtful value.

A careful reading of the paper will probably make plain any apparent ambiguity in regard to compression in members subject to alternate strains. Mr. Greiner did not state whether the required section found by addition would be the necessary net or gross area.

The opinion expressed against chemical analysis is not in conformity with the present practice. No short-time physical tests can be made which will detect micro-flaws due to high sulphur. How does Mr. Greiner provide for this, or does he consider it the invention of "theorists"?

The opinion in favor of drifting tests is directly contrary to that held by the late James G. Dagron, M. Am. Soc. C. E., who, for many years, was Chief Inspector of the Baltimore and Ohio Railroad, and later Engineer of Bridges of that road.

Mr. Seaman. In Mr. Greiner's statement that the specifications do not provide against bad details, he has probably overlooked the clause which requires all working drawings to be submitted for the approval of the railroad company.

The series of quotations cited by Mr. Wilson in corroboration of the 2 to 1 ratio is most timely and pertinent, and is the strongest endorsement possible for this ratio. When an examination of the fatigue experiments indicates a corresponding ratio, it demonstrates that this item may be included in the same proportion. It is certainly the simplest ratio conceivable and should require the strongest evidence of error to justify its abandonment for any other method of proportioning.

When Mr. Cooper questions fatigue, but adopts the 2 to 1 ratio to cover "contingencies unknown"; and others, realizing the existence of fatigue, note that it should be provided for in the same ratio, the margin of difference is very slight.

The increased impact on short span has long been recognized, and recent experiments give more definite light upon this subject, though much yet remains to be learned.

The tables published by Mr. Wilson are interesting records of the method originally adopted by him in the use of the Launhardt formula. It is doubtful if there is any column table adopted since this, which is as quickly applied to new work.

It is unfortunate that a difference should have arisen as to the equivalent of the German centner per square inch, because it does not affect the comparative results, and tends to confuse a subject already somewhat abstruse.

By *Engineering*, 1 centner per square inch (German) = 107 lbs. per square inch (English).

By Professor Marburg, 1 centner per square inch (German) = 104 lbs. per square inch (English).

It appears that the German inch is itself variable in the different German kingdoms, and that Professor Marburg, after careful investigation, has probably reached the correct value, but to avoid confusion in the discussion the author has for the present paper adhered to the value given by *Engineering*.

The criticism that the results on wrought iron were obtained by strains of different kind is correct, as shown on page 142, but the statement that the formula for transverse strain does not apply to strains beyond the elastic limit, is irrelevant, since the formula was only used where the strains were within that assumed limit. The value of 300 centners (32 100 lbs.) which Professor Marburg uses, does not conform to the published tests; for though 32 100 lbs. (300 centners) was applied indefinitely, so also was 34 240 lbs. (320 centners). It is true that the tension tests of 34 240 lbs. resulted

in rupture, but this was only after 10 141 645 repetitions, which Mr. Seaman would at least indicate that it was very close to the correct value. The value of 47 080 lbs., 25 680 lbs. was carried only to 4 000 000 repetitions. It is not clear, therefore, why these values are not truly comparative.

Professor Marburg has introduced Groups III and IV, but states that data were not available by which he could plot Group III in Fig. 5. It was for a similar reason that the author excluded these groups altogether. They are not sufficiently complete to prove that they are not abnormal, and are of little value in comparison with Group II. The author's purpose in the 2 to 1 ratio was more especially in comparison with the Launhardt formula. In the application of this ratio to bridge construction the element of fatigue becomes only a partial factor.

It is not clear why the fourth and fifth items of Group I, Table No. 5, are added, since they are not included in Wöhler's duration tests. The same criticism applies to the fifth and sixth items of Group II.

Fig. 5 shows a noticeable uniformity, even in the discrepancy of the tests from the line *c b* and would approximate a line running from $5\frac{1}{2}$ at *c* to 11 at *b*. It is to be regretted, however, that the paper should have been misconstrued as advocating a precise interpretation, even of the 2 to 1 ratio, as it was this very precision which the author wished to avoid, and believed he had disproved. It was expressly stated that the experiments were "too incomplete to be considered as a demonstration of an exact law," and the "deductions were by no means conclusive." but "the ratio receives much more confirmation than does the Launhardt formula." This last statement remains untouched.

Professor Marburg evidently regards the theory of work, as one to be measured by force \times distance, since he considers the records of deformation as necessary to that theory. The word "work," however, was used in the sense of an exertion of strength, whether it be physical work, brain work, or other form of energy.

There appears to be some difference between the interpretation of Bauschinger's results by Professor Marburg and by the author. It is unfortunate that no authorized translation has yet been made, but Professor Marburg has evidently failed to grasp the difference between the "elastic limit" as formerly understood, and that which has been modified to meet Bauschinger's experiments. According to the author's memoranda these experiments showed that the old definition of elastic limit, based upon the absence of permanent elongation must be abandoned from the beginning, since the permanent elongation is never entirely eliminated after a strain is once applied. If the term "elastic limit," or better, "elastic strength," is retained, it must be with Bauschinger's new definition, as limit of uniform elongation.

Mr. Seaman. Bauschinger also found a second limit which he termed the "limit of strength" (*streck grenze*). Within this second limit the elongation does not increase under continued action of a static load; nor does it diminish after the load is relieved. Beyond this second limit the elongations increase under static load, during several hours or possibly days; and after the load is removed, the elongations decrease.

Bauschinger made some very interesting deductions, too profuse for insertion here, and some of which may perhaps be questioned in view of experiments made by others, both before and since that time, but they throw new light upon Wöhler's tests and their practical application. Among the most pertinent of his deductions are those which describe the almost infinite number of "elastic limits" which may develop under various conditions, and also that which explains that when the "elastic limit" is exceeded by strains in one direction, it is at once completely destroyed for strains in the opposite direction; thus possibly explaining the fact described by Mr. Morison, that columns subject to compression apparently fail by buckling in tension. Bauschinger's statement that when a piece has been subject to strains repeated many times, the resistance to rupture under static load is not diminished by them, but rather is increased, is also of the greatest importance, in view of the effort made by some, to limit the dead strain to values "well within the elastic limit."

If the chief interest in Wöhler's experiments were found in the fact that they proved "(a) that innumerable repeated stresses of like character, not exceeding the primitive elastic limit, cannot produce rupture, and (b) that the safe limit for equal stresses, opposite in character, is far below the primitive elastic limit," their value would be limited indeed. These facts have long been recognized and provided for. The real value of Wöhler's experiments and of those of Bauschinger, which followed, lies in the fact that they show approximately what may be expected beyond the "elastic strength," and thus make possible an additional factor of safety and economy in bridge design.

The author does not share the regret, kindly expressed by several, that the prevailing tendency of discussion is to criticise. Such a discussion by men who are recognized authorities upon the subject of which they write is probably the most severe test to which a proposition can be subjected, and is a wholesome restraint upon radical and purposeless innovations.

The specifications in Appendix B have been revised by the light of the discussions received.

APPENDIX B.

SPECIFICATIONS FOR STEEL RAILROAD BRIDGES (REVISED.)

PROPOSALS AND DRAWINGS.

1. Each bidder shall submit with his proposal complete strain Preliminary sheets, showing loads assumed in calculations and the resulting strains, and the sections used. The strains from each kind of load shall be shown separately, and the dead load assumed shall not be less than that of the finished structure. At the same time, each bidder shall submit general plans of the structure, and such details as will show clearly the construction of all members and connections. In case of draw bridges, all machinery shall be similarly shown.

2. All drawings furnished by the railroad company, whether of general location or of details of construction, shall be strictly followed.

3. Upon the award of the contract and before work is commenced, a complete set of working drawings, in duplicate, including strain sheets and general drawings previously mentioned, shall be submitted to the railroad company for approval. All material ordered or work done before the drawings are approved shall be at the risk of the contractor. Drawings approved.

4. All drawings shall be made on the dull side of tracing cloth, and of a uniform size of 24 x 36 ins. After the work is completed, these drawings, in good condition, shall become the property of the railroad company for file. The contractor may retain such prints or copies of them as he may desire for record. Uniform sheets.

GENERAL PROVISIONS.

5. The structure shall be wholly of rolled steel and wrought iron. A type of truss shall be used in which the strains may be readily calculated and which subjects no pin-connected member to alternate strains. Continuous girders will be allowed only in case of upper chords carrying floors, and in special cases of draw bridges. Type of truss.

6. Double-track, through-truss bridges shall have only two trusses, and four-track bridges only three trusses, unless otherwise specified. In the case of plate girders, special provisions shall be made for spreading the tracks when necessary.

7. All through pin-connected bridges shall have floor beams connected by a web, with both segments of the posts. Hip verticals shall be made rigid. All spans shall have rigid end panels of lower chord when required.

8. When in deck-plate girders the variation in thickness of upper flange plates exceeds 1 in., a separate floor system of stringers shall be provided to carry the ties.

Clearance. 9. The distance from center to center of double track is 13 ft. *

10. All through-bridges, on tangents, shall have a clear opening as shown on the clearance diagram, Fig. 7, or as may be required by local State laws. The width shall be proportionately increased for two or more tracks. On curves, the height and width shall be increased as required by curvature and elevation of rail.

Floor system. 11. All bridges shall be provided with a steel floor system. Stringers and deck-plate girders shall be spaced a distance apart between centers equal to their depth, with a minimum limit of 6 ft. 6 ins., unless otherwise required.

Wooden floor. 12. The wooden floor of ties and guard rails will be furnished and put on by the railroad company, unless otherwise specially provided by contract. The cross-ties shall be of long-leaved yellow pine, 8 ins. square and 9 ft. long, spaced 6 ins. apart in the clear and notched $\frac{1}{2}$ in. over the stringer. In cases where the stringers are spaced more than 6 ft. 6 ins. apart, the depth of the tie shall be increased so that the strain on the outer fiber does not exceed 1 000 lbs. per square inch, considering the weight of a single driver as being carried by two ties.

13. Guard rails of long-leaved yellow pine, 6 x 8 ins. shall be placed 3 ft. 9 ins. in the clear, from the center of the track. They shall be notched $\frac{1}{2}$ in. over the cross-ties and spliced with a horizontal half and half joint, 6 ins. long over a tie. They shall be bolted to the cross-tie at the splice, to the cross-tie next adjacent to the splice, and to every fourth tie between, by $\frac{3}{4}$ -in. bolts. To all other ties they shall be fastened by $\frac{3}{4}$ -in. square spikes. Inside rail-guards shall be placed at a distance of 7 ins. in the clear inside the track rail, and shall extend back over the approach to a point not less than 50 ft. from the back wall.

LOADS.

14. The structure shall be designed to resist the strains from the following loads:

Dead load. 15. The dead load shall consist of the entire weight of the structure, properly distributed at the various panel points.

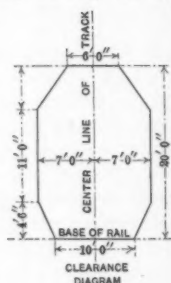


FIG. 7.

* This distance may vary according to the practice of different roads.

16. The weight of rails, guard rails, splices and bolts shall be estimated at 175 lbs. per lineal foot of track: ties of standard dimensions at 225 lbs. per lineal foot of track, and special ties at $4\frac{1}{2}$ lbs. per foot B. M.

17. The live load shall be the moving load, with impact. The moving load shall consist of two typical consolidated engines, followed by a train, and distributed as shown in the diagram, Fig. 8.

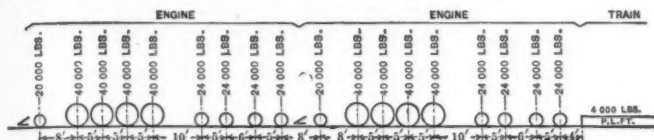


FIG. 8.

18. There shall be as many trains as tracks, each placed in such position as to produce the maximum strains in the structure.

19. The impact will be the increase of live strain, due to the sudden application of the moving load, and shall be provided for according to the following table by interpolation:

L (ft.)	0	10	20	30	40	50	60	70	80	100
I (%)	50	28	20	14	10	7	4	2	1	0

Where L = Distance in feet through which the moving load must pass to produce the given strain.

I = Percentage of increase of moving load.

20. When the bridge is on a curve, the lateral bracing and vertical trusses shall be proportioned to resist the centrifugal force due to as many trains as there are tracks, moving in the same direction, at the rate of 60 ft. per second.

21. Provision shall be made for the sudden starting or stopping of trains, estimating the coefficient of sliding friction at 20 per cent.

22. Provision shall be made for wind pressure, acting in either direction horizontally of 40 lbs. per square foot of the surface of all trusses and the floor, as seen in elevation, considering the ties solid area; in addition to a train 10 ft. high, beginning 2 ft. 6 ins. above base of rail, and moving across the bridge. In the case of plate girders, only one girder need be considered. The wind forces shall be properly distributed between the upper and lower chords.

23. Provision shall be made for a variation of temperature of 150° Fahrenheit.

ALLOWABLE STRAINS.

24. The live strain will be the total variation in strain produced by the live load. The dead strain will be the minimum strain to which a member is subjected.

25. The allowable live strain per square inch shall not exceed one-half of that permissible for dead strain.

26. The allowable dead strain per square inch for rolled steel shall not exceed 18 000 lbs.

27. When wrought iron is used, the allowable dead strain per square inch shall not exceed 16 000 lbs.

Columns. 28. For columns subject to direct compression only, the allowable working strain of 18 000 lbs. per square inch shall be reduced, in proportion to the ratio of length to least radius of gyration, by the following formula:

$$p = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{18\,000 r^2}}$$

Where p = allowable dead strain per square inch, in pounds,

l = length of column, in inches,

r = least radius of gyration of cross-section, in inches.

29. For columns subject to alternate strains of tension and compression, the compressive strain shall be increased to provide for the increase of strain due to flexure in proportion to the ratio of the length to the least radius of gyration by the formula:

$$P = p \left(1 + \frac{l^2}{18\,000 r^2} \right)$$

Where p = direct compression in member,

P = increased strain due to length of column,

l and r = length and least radius of gyration of cross-section, in inches.

30. The increased compressive strain per square inch (gross) thus found shall be added to the tensile strain per square inch (net), and the resulting total unit strain shall not exceed that specified for live strain.

Compression flanges. 31. In the case of compression flanges of beams and girders, the allowable working strain per square inch of such flanges for dead strain shall be computed by the formula:

$$C = \frac{18\,000 \text{ lbs.}}{1 + \frac{l^2}{5\,000 w^2}}$$

Where C = allowable compressive strain per square inch,

l = unsupported length of compressed flange, in inches,

w = width of flange, in inches.

Strains on pins, rivets, etc. 32. The shearing on pins, rivets, and bolts shall not exceed for dead strain 13 500 lbs. per square inch of cross-section. Where tension on rivets is unavoidable, it shall not exceed one-half the limit allowed for direct shear. When a force is oblique, the components of

direct tension and of direct shear shall be considered separately and the results combined.

33. The bending strain in the outer fiber and the bearing pressure (diameter \times thickness) on pins, rivets and bolts, shall not exceed for dead strain 27 000 lbs. per square inch.

34. In cases of field rivets, driven by hand, an excess of 25% shall be allowed.

35. Wind pressure and sliding friction, either separately or combined, shall not produce greater strain per square inch than that allowed for dead strain. Centrifugal force shall be considered as live strain. Lateral strains.

36. The pressure, in pounds per lineal inch of roller, shall not exceed $1\ 200 \sqrt{d}$ for dead load (d = diameter, in inches). Rollers.

37. Bed plates on masonry shall be so proportioned that the greatest pressure on the masonry does not exceed 300 lbs. per square inch. Bed plates.

DETAILS OF DESIGN.

38. The assumed spans for calculation shall be as follows: Span for calculation.
 For pin-connected trusses—distance between centers of end pins.
 “ riveted girders “ “ “ bearing.
 “ cross-girders “ “ “ trusses.
 “ track stringers “ “ “ of cross-girders.
 “ cross-ties “ “ “ of track stringers.

39. The assumed depth for calculation shall be: Depth for calculation.
 For pin-connected trusses—distance between centers of chord pins.
 “ riveted lattice girders “ “ “ “ gravity of
 flanges (not to exceed the distance out to out of angles).
 “ plate girders—distance out to out of angles.

40. The whole of the wind force due to the train and floor, and one-half of the truss, shall be considered as acting on the lateral system of the loaded chord, and that due to one-half the truss only, on the lateral system of the unloaded chord. Lateral bracing.

41. In the case of deck bridges and very heavy curves, some of the centrifugal force may be transferred to the lower lateral system, in which case the truss shall be duly strengthened. The end portal bracing in through bridges must be of sufficient strength to transfer the accumulated wind strains from the upper lateral system to the end posts, and the end sway-bracing in deck bridges shall carry the whole of the accumulated wind and centrifugal forces from the loaded chord to the abutment.

42. Each main panel of deck bridges shall be provided with intermediate sway-bracing of a sufficient section to carry one-half the maximum increment due to wind on train, and to centrifugal force. Through bridges shall be provided with post brackets at the intermediate panel

points, of sufficient strength to maintain the panel in a vertical position under the specified wind pressure, or, when the height of top chord exceeds 25 ft. above base of rail, an overhead system of sway-bracing shall be used.

43. Lateral and sway-bracing shall be designed to receive an initial tensile strain in the diagonals of 10 000 lbs. or shall be capable of sustaining a compressive strain of one-half that amount.

44. Tension at the windward column of trestle piers shall be avoided if possible, and in any case approved anchor bolts well secured to the masonry shall be used.

45. The struts shall be proportioned to withstand their component of strain. No reduction shall be made from chord section on account of the material in the lateral system.

Design of
details.

46. All parts shall be so designed that the strains coming upon them may be definitely calculated. The center line of resistance of a member will be along its neutral axis, and connections shall be so designed as to avoid bending, twisting or unequal tearing of the member or its details. The line of strain shall pass centrally through any cluster of rivets which resist it, and where angles or plates are otherwise than so connected, proper provision shall be made for the moments and secondary strains produced. Details shall be so designed as to give free access for inspection and painting, and water pockets shall be avoided. In every case the connection of details shall be of greater strength than the member itself.

47. All members which are subject to direct strains, in addition to bending moments, shall be so proportioned that the algebraic sum of the strains coming upon them shall not exceed the specified allowable strain, properly reduced in case of columns. In continuous upper chords of deck bridges, carrying the floor, the strain due to the live load shall be computed from a bending moment equal to $\frac{2}{3}$ of the maximum moment produced by the engine on a span equal to a panel length considered as a simple beam.

48. The strain on the outer fiber of solid shapes shall be computed from the moment of inertia of the section.

Plate
girders.

49. No allowance shall be made for the web in calculating the flange section of plate girders. Girders formed of web plates and angles alone, having no upper flange plate proper, will be allowed only when ties rest upon the upper flange. At least one upper flange plate, when used, shall extend from end to end of the girder, and any additional plates used to make up the flange section shall be made of such lengths as to allow at least two rows of rivets of the regular pitch being placed at each end of the plate beyond the theoretical point required, and there shall be a sufficient number of rivets at the ends of the plates to transmit their value before the theoretical point of the next outside plate is reached. Where the flange plates vary in thickness, they shall decrease

outward from the flange angles. All plate girders shall have end and corner cover plates. The total thickness of plates and angles shall not exceed five times the diameter of the rivet used.

50. All flange plates, subject to either tension or compression, spliced in the length of the girder, shall be covered with an extra amount of material equal in section to the material spliced, with sufficient rivets on either side to transmit the strains from the parts cut. Flange angles shall be spliced with angle covers.

51. In calculating the shearing or bearing strain in web rivets of plate girders, the whole of the shear acting on the side of the panel next to the abutment shall be considered as being transferred into the flange angles at a distance equal to the depth of the girders.

52. The webs of plate girders shall be spliced, wherever cut, by a plate on each side of the web capable of transmitting the full shearing strain through splice rivets.

53. When the thickness of the web plate is less than $\frac{1}{30}$ of the unsupported distance between flange angles, stiffeners shall be riveted on both sides of the web, with a close bearing against the upper and lower flanges and calculated as columns by the compression flange formula for the whole shear at the several points where they are placed. The distance, center to center, of stiffeners, shall generally not exceed the depth of the full web plate, but shall not be less than 4 ft. Web plates generally shall have stiffeners at all splices, at points of concentrated loading, and at each end of bearing plates.

54. Net sections shall be used in all cases in calculating tension members, and in deducting rivet holes they shall be taken as $\frac{1}{8}$ in. wider than nominal diameter of rivet. In calculating the net sections, having rivets staggered, all rows shall be deducted, unless so arranged that the net section along a zigzag line, taking all distances in the diagonal direction at only three-fourths their value, exceeds the corresponding net section directly across the plate. Net sections.

55. Rivets shall not be spaced closer than three diameters, center to center, nor further apart, in the direction of the strain, than sixteen times the thickness of the thinnest external plate connected, and not more than thirty times that thickness at right angles to the line of strain. Rivet spacing.

56. Rivets shall not be spaced closer to the side of the plates than $1\frac{1}{2}$ diameters to the center of the rivet, nor further from the side than eight times the thickness of plate. In no case shall the pitch of rivets exceed 6 ins.

57. Field rivets shall be reduced to a minimum.

58. Built chords shall be thoroughly spliced with rivets and additional section sufficient to transmit the entire strain; no allowance shall be made for abutting surfaces, except in work exceeding $\frac{3}{4}$ in. in thickness. Chord splices.

Pin plates. 59. When necessary to obtain sufficient bearing surface at pin-holes, re-enforcing plates shall be added. These plates shall distribute the bearing strains from the pins to the member to which they are connected.

Compression members and latticing. 60. All segments of members in compression, connected by latticing only, shall have batten plates at each end, the thickness of which shall not be less than $\frac{1}{40}$ of the distance between rivets connecting them to the compressed member. In no case shall the length of the batten plate be less than $1\frac{1}{2}$ times the width of the member.

61. The distance between connections of latticing shall be such that the individual members composing the column, considered with hinged ends and a length equal to the distance between these connections, shall be stronger than the column as a whole, and in no case shall this distance exceed eight times the least width of these members. Where the ends of the compression member are forked to connect the pins, the strength of each leg shall be at least equal to the entire strength of the column, and the re-enforcing plates shall extend not less than 6 ins. beyond the edge of the batten plates.

62. Single lattice bars shall have a thickness of not less than $\frac{1}{40}$, and double lattice bars not less than $\frac{1}{30}$ of the distance between the rivets connecting them to the compressed member, and their widths shall be:

For—

33 ins. and over, web,	4-in. angles	($2\frac{7}{8}$ -in. rivets),	3 x 2 x $\frac{3}{8}$ -in. angles.
29 ins. to 32 ins.,	" "	" "	$2\frac{1}{2}$ x $2\frac{5}{8}$ -in. "
20 ins. to 28 ins.,	" "	" "	4-in. straps.
15 ins. to 19 ins.,	" "	($1\frac{7}{8}$ -in. rivets),	4-in. "
12-in. channels, or 3-in. angles,	" "	" "	$2\frac{3}{4}$ -in. "
9-in. "	" $2\frac{1}{2}$ -in. "	($1\frac{3}{4}$ -in. rivets),	$2\frac{1}{2}$ -in. "
8-in. "	" 2-in. "	" "	2-in. "

63. Single lattice bars shall generally be inclined at an angle of 60° to the axis of the member, and double lattice bars at an angle of 45° , with a rivet at their intersection.

Expansion rollers. 64. All bridges over 80 ft. long shall be provided at one end with turned friction rollers, not less than 3 ins. in diameter, between two planed surfaces. For spans of 80 ft. or less, planed surfaces shall be used without rollers. The nest of rollers shall be easily cleaned from the accumulation of dust and cinders.

Bearings anchored. 65. Trusses shall be secured against side motion on bearing plates and rollers. The bolster blocks shall be joined to the truss, and the bearing plates shall be secured to the underlying supports by bolts or dowels.

Eye-bar packing. 66. Eye-bars shall be so packed as to produce the least bending moment on the pin, and shall not be packed out of line with the axis of the member more than $\frac{1}{8}$ in to 1 ft.

67. No iron less than $\frac{3}{8}$ in. thick shall be used, except for packing or other idle material. No counter rod shall have less than $1\frac{1}{2}$ sq. ins. of sectional area. Minimum section.

68. The camber shall be such that under maximum load the bridge will not deflect below a horizontal position. Camber.

QUALITY OF MATERIAL.

69. *Rolled Steel.*—Rolled steel shall be made by the open-hearth process, and shall contain not more than 0.04% phosphorus, 0.04% sulphur, nor 0.45% manganese. The steel shall be finished straight and smooth, and shall be a perfect product; the slightest flaw will be sufficient cause for rejection at any time during the progress of the work. Chemical analysis and finish.

70. The tensile strength, yield point, and ductility of the material shall be determined from a standard test piece of not more than 2 ins. in width, nor less than $\frac{1}{4}$ sq. in. in sectional area, cut from a full-sized bar, and with sides turned or planed parallel, so as to give a uniform minimum section for a length of at least 12 ins. Standard test piece.

71. Whenever practicable, the two sides of the test piece shall be left as they come from the rolls, but the finish on opposite sides shall be alike in this respect.

72. In determining the ductility, the elongation shall be measured after breaking, on an original length of 8 ins., in which length shall occur the curve of reduction each side of the point of fracture.

73. The yield point shall be that strain beyond which the elongation ceases to be proportional to the weight imposed, and may be indicated by "drop of beam." It shall in no case be less than 55% of the maximum strain sustained by the test piece. The speed of testing shall be governed by the inspector. Yield point.

74. All rolled steel, except rivet steel, shall show by the standard test piece a maximum strength per square inch of 56 000 lbs. \pm 4 000 lbs., with an elongation of 26% in 8 ins. Rivet steel, when tested in specimens of full size of rivet rod, shall show an ultimate strain per square inch of 52 000 lbs. \pm 4 000 lbs., with an elongation of 30% in 8 ins. Maximum strength and elongation.

75. Each melt of finished material shall receive two tension tests—one cut from each extreme variation in thickness of metal rolled. When both tests comply with the specifications, all intermediate thicknesses will be accepted; otherwise only such thicknesses of metal will be accepted as show satisfactory tests.

76. Each finished piece of steel shall be marked with the melt number. Material marked.

77. *Wrought Iron.*—All wrought iron shall be tough, ductile, fibrous and uniform in quality. It shall be thoroughly welded in rolling, and finished straight and smooth. It shall be free from flaws, blisters, Finish.

cinder-spots, cracks and imperfect edges. Scrap steel shall not be used in its manufacture.

78. The methods specified for testing rolled steel shall apply generally to wrought iron.

Maximum strength and elongation. 79. All iron shall show by the standard test piece a maximum strength of not less than 50 000 lbs. per square inch and an elongation of 20 per cent.

Yield point. 80. The yield point, as shown by the standard test piece, shall in no case be less than 26 000 lbs. per square inch.

Bending tests. 81. All iron when cut into testing strips $1\frac{1}{2}$ ins. in width and with corners rounded to $\frac{1}{8}$ in. radius, must be capable of resisting, without signs of fracture, bending cold 90° with the inner radius not to exceed three times the thickness of the test piece.

82. All iron which is to be bent in manufacture shall, in addition to the above requirements, be capable of bending sharply to a right angle at a working heat without any signs of fracture.

83. *Cast Steel*.—Steel castings shall be true to pattern and free from injurious blow holes or other imperfections. Coupon specimens, turned to $\frac{3}{4}$ in. diameter, shall show an ultimate strength per square inch of 65 000 lbs., a yield point of 35 000 lbs., and an elongation of 15% in 2 ins.

84. *Cast Iron*.—All cast iron shall be tough and sound, free from blow-holes, cold-shuts or other injurious imperfections. When broken, the fracture shall indicate a good quality of gray iron.

85. Sample test specimens, 27 ins. long, 2 ins. \times 1 in. in cross-section, cast under the same circumstances as those which attend the casting of the full-sized piece, shall sustain at the center, when resting flatwise upon two dull knife edges, spaced 24 ins. apart, a load of 2 000 lbs.; the load to be sustained two minutes, and show a deflection of not less than $\frac{1}{4}$ in. before fracture.

WORKMANSHIP.

Finish. 86. All workmanship shall be first class. All parts exposed to view shall be neatly finished. All nuts shall be hexagonal.

Punching. 87. In punching, the diameter of the punch shall not exceed by more than $\frac{1}{16}$ in. the diameter of the rivet to be used, and the diameter of the die shall be as small as may be required to punch a clean hole.

88. The holes shall be so carefully spaced and punched that, upon assembling, no variation from a truly opposite position of more than $\frac{1}{16}$ in. will occur. All holes shall be reamed to an exact match before the rivet is driven.

Rivets. 89. Rivets when driven, shall completely fill the holes and shall be machine driven whenever possible. They shall have full, con-

centric, hemispherical heads of a depth at circumference of shank of not less than one-half the diameter of rivet, and with full bearing on the plates; or they shall be countersunk when so required. Rivet heads shall not be flattened to less than half the diameter of the rivet unless countersunk.

90. Generally, the use of bolts instead of rivets will not be permitted, but when used in special cases the holes shall be reamed parallel, and the bolts turned to a driving fit. Turned bolts.

91. All holes for field rivets, excepting those in connections of lateral and sway bracing, shall be accurately drilled to an iron template, or reamed while the connecting parts are temporarily assembled in the shop. In the case of splices of upper chords, or other compression members, they shall be brought to forcible contact by the use of turnbuckles, and after reaming shall have match marks put on the pieces so that they may be brought to proper position in the field, before riveting. Template reaming.

92. Finished members shall be true and free from kinks, twists and open joints. Ends of floorbeams shall be finished square and true. Members true.

93. Where rivets may take tension, the rivet holes shall have edges rounded to $\frac{1}{8}$ in. radius. Tensile rivets.

94. Rods and bars which are to receive a thread shall be properly upset. Where threads are cut on steel, they shall be properly filleted. Screw threads.

95. All members requiring adjustment shall be provided with sleeve nuts and check nuts. Open turnbuckles will not be allowed. The ends connected shall be distinctly punch-marked, at a distance of 12 ins. from the screw ends, so that these ends may be accurately located inside the nut. Adjustable members.

96. Heads of eye-bars shall be of sufficient section to break the bar in every case. Bars shall be full size at the neck; no patching will be allowed. Eye bar heads.

97. Welds in the body of the bars or rods will not be allowed.

98. Pins shall be turned, perfectly finished, and straight.

99. All members having bearing on pins shall be carefully bored at right angles to the axis, unless otherwise shown in the drawings. No variation will be allowed between diameter of pin and pin hole of more than $\frac{1}{16}$ in. For pin holes, in pieces which are not adjustable for lengths, no variation of more than $\frac{1}{8}$ in. in length between centers of pin holes will be allowed. Boring.

100. Eye-bars shall be perfectly straight before boring; the holes shall be in the center of the heads and on the center line of the bar. Bars which belong to the same member shall be bored at the same temperature and in one operation. They shall be marked for erection, so that they may be used in the same member.

- Abutting surfaces. 101. All abutting surfaces, except flanges of plate girders, shall be neatly planed or turned perpendicular to the direction of the strain, so as to insure even bearings.
- Stiffeners. 102. Stiffeners of plate girders shall be milled to fit tightly against the flange angles, and shall be packed straight. The packing shall fit close to the flange angles, leaving no open space.
- Rollers turned. 103. Rollers shall be turned and roller beds and bed plates planed; the bottom of shoes shall be planed exactly parallel to the center line of pin, unless otherwise shown in the drawings.
- Thickening washers. 104. Thickening washers shall be used whenever required to pack the pin joints tight.
- Pilot nuts. 105. Pilot nuts shall be furnished for each size of pin to preserve the thread of the pin, and to facilitate erection.

PAINTING.

106. All iron shall be scraped free from scale, and receive one coat of pure kettle-boiled linseed oil before leaving the shops, and one coat of approved paint after erection. All surfaces which come in contact, or are enclosed, shall receive one coat of approved paint before being assembled.

107. All turned or faced surfaces shall receive a coat of white lead and tallow before leaving the shops.

INSPECTION.

108. Free access and information shall be given by the contractor for a thorough inspection of material and workmanship.

109. The inspector will make detailed reports of his inspection to the engineer and may notify the contractor of any defects in material and workmanship, but all acceptances made by him shall be considered as temporary, and his inspection shall in no way relieve the contractor of full responsibility for the character and accuracy of the work until its completion and final acceptance by the engineer.

110. The contractor shall furnish without extra charge such standard test pieces as may be necessary to determine the uniform quality of the material, and also the use of a reliable testing machine with necessary labor for testing.

111. Full-sized members may be tested to destruction by the engineer. They will be paid for at cost, less their scrap value, if they fulfill the requirements, but when the requirements are not complied with, the tests shall be at the expense of the contractor. Such rejections of the finished material as he may consider warranted by the results of these tests will be made by the engineer.